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Enquiries pertaining to the Canada-Ontario Agreement RESEARCH PROGRAM should be directed to -

Wastewater Technology Centre,
Canada Centre for Inland Waters,
Environment Canada,
P.O. Box 5050,
Burlington, Ontario L7R 4A6

Ontario Ministry of the Environment,
Pollution Control Branch,
135 St. Clair Avenue West,
Toronto, Ontario M4V 1P5

REVIEW OF CANADIAN DESIGN PRACTICE AND COMPARISON OF URBAN HYDROLOGIC MODELS

by

James F. MacLaren Ltd.

RESEARCH PROGRAM FOR THE ABATEMENT
OF MUNICIPAL POLLUTION WITHIN THE
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M4V 1P5

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ABSTRACT

A questionnaire was distributed to municipalities across Canada requesting details of their storm sewer design practice with emphasis on policies, problems with flooding and frost conditions, and attitudes towards new trends in urban drainage management. Replies from 37 municipalities from Vancouver to Halifax were received and analysed.

Five urban hydrologic models were evaluated. These were:

1. The Storm Water Management Model (SWMM),
2. The University of Cincinnati Model (UCUR),
3. The Transport and Road Research Model (RRL),
4. The Queen's University Urban Runoff Model (QUURM),
5. The Proprietary Dorsch Consult Hydrograph Volume Method Model (HVM).

These models were evaluated using recorded data from 4 residential test areas from the United States and Canada, using from 10 to 14 different storm events on each area. The accuracy and consistency of the models was assessed by means of graphical and statistical comparison of both peak flows, times to peak, runoff volumes and the complete hydrograph for the computed and recorded hydrographs.

The peak flows calculated by the Rational Method were also compared with the measured flows.

It was found that all models performed satisfactorily when applied to small test areas and that all of the models gave more consistent results than the Rational Method.

RÉSUMÉ

Un questionnaire, a été envoyé a plusieurs municipalités à travers le Canada, dans lequel on demandait des détails sur le mode de conception des égouts pluviaux dans le contexte des politiques adoptées en ce sens, des problèmes occasionnés par les crues et le gel, et des attitudes prises face aux développements nouveaux dans le domaine de la gestion des systèmes d'égouts urbains. Des réponses provenant de 37 municipalités de Vancouver à Halifax ont été analyseés.

Cinq modèles hydrologiques urbains ont été étudiés:

1. modèle de gestion des eaux de pluies (SWMM);
2. modèle de l'Université de Cincinnati (UCUR);
3. modèle du Transport and Road Research Laboratory (Angleterre) (RRL);
4. modèle de l'écoulement de l'eau en milieu urbain de l'Université Queen (QUURM); et
5. modèle appartenant à la Dorsch Consult et fondé sur une méthode par hydrogrammes et volumes (HVM).

Ces modèles ont ete évalués à partir de données provenant de 4 régions résidentielles des États-Unis et du Canada où, pour chacune on a tenu compte de 10 à 14 événements associés à des orages. La précision et la concordance des modèles ont été évaluées par comparaison graphique et statistique des débits de pointe, des temps d'écoulement maximal, des volumes des écoulements et par un hydrogramme comparatif pour les données calculées et enregistrées.

Les débits de pointe calculés par la méthode rationnelle ont aussi été comparés aux débits mesurés.

On a observé que tous les modèles étaient satisfaisants quand ils étaient appliqués à des régions peu étendues, et que tous donnaient des résultats plus cohérents que la méthode rationnelle.

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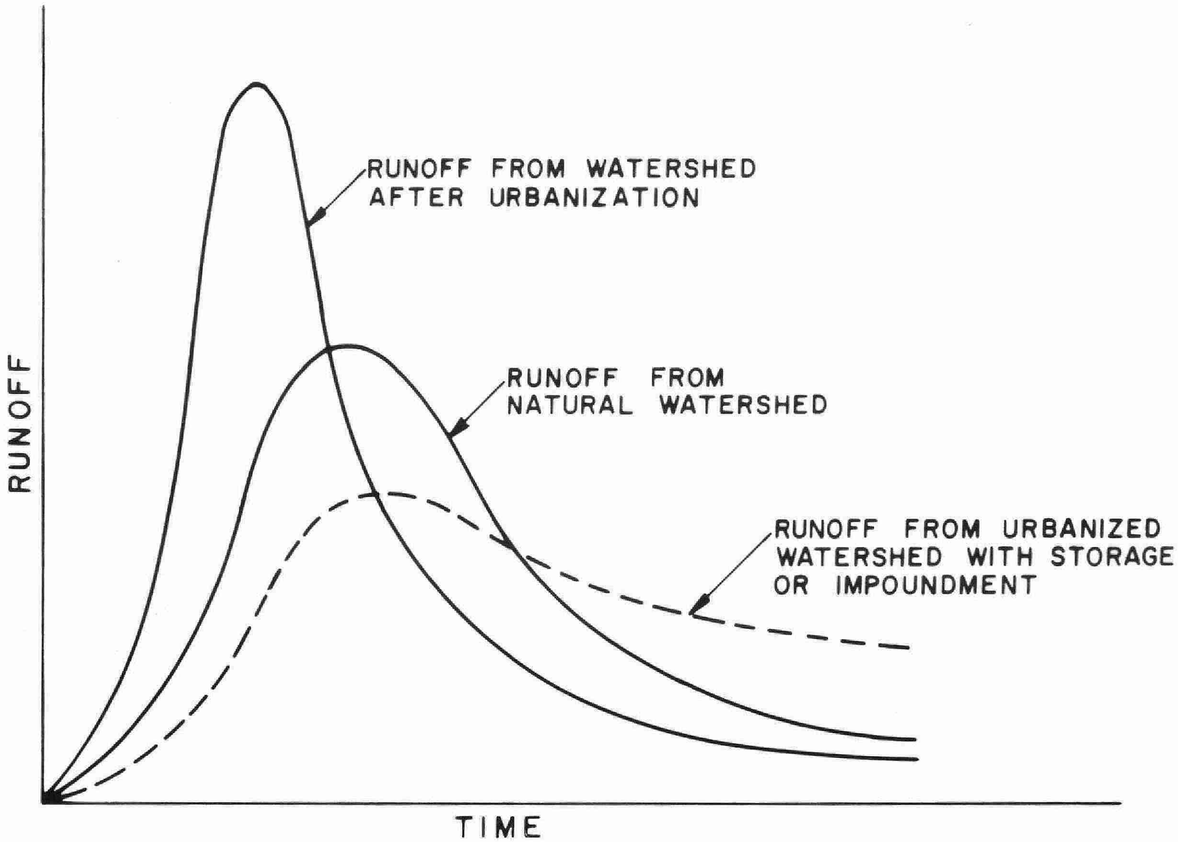
1. INTRODUCTION

Storm Water Management, considering drainage as a sub-system of the total urban system with environmental aspects and possible benefits, is a relatively new concept.

The traditional storm drainage design philosophy was to collect the runoff and carry it away as fast as possible out of the boundaries of the considered watershed. This was done by connecting all impervious areas such as roofs and driveways to a network of gutters and conduits with considerably higher velocity and density than in the natural drainage system. Storm water was also considered "clean" and there was no concern with regard to pollution from separate storm sewers. The design of the storm sewer system was carried on independently from the studies for flood control from rare events. Negative consequences of this philosophy, such as dramatic increase of the peak flows at the outlet of the urbanized watershed, increased incidence of local flooding, depletion of ground water, considerable increase in the cost of new storm sewerage systems and relief sewers, and environmental damage are now evident and many attempts for an innovative approach are underway.

The key to the implementation of new management methods, however, is the use of improved hydrologic tools. The design of storage, for example, which is the simplest method for reduction of flow peaks is possible only through the synthesis of hydrographs (Figure 1). Storage in an urban system is not necessarily concentrated in a reservoir but may be distributed over different elements of the watershed such as parking lots, roofs, elements of the sewer network, etc. Other methods of peak reduction are the retardation of flow by reduction of velocity or increase of infiltrated volumes. The traditional design method for drainage systems, the Rational Method, is aimed at providing only design peak flows and cannot be used for the study of management techniques. Even the use of the Rational Method for the derivation of design peak flows has been subject to numerous criticisms.



EFFECT OF URBANIZATION AND STORAGE ON RUNOFF HYDROGRAPHS

Fig. 1

Therefore, an increasing number of more sophisticated models dealing with urban storm water runoff, some of which include quality considerations, are being developed. A list of recent models is given in Table 1. The selection of a model, even if only runoff is considered, is obviously a difficult task. Many of the new models are at an initial or developmental stage and most of the references regarding proprietary models give only limited information. During this study it was found that, even for non-proprietary models, the selection of appropriate models required the assistance of the model builder or consultants with previous experience in their application and, eventually, some modifications to the models as well. Comparative analyses of the models prior to the beginning of this study were limited, and the adequacy of the most easily available models was determined by only a few comparisons with measurements. These conditions add to the confusion created by different publications advocating one model or another and could slow down the implementation of new storm water management techniques.

In order to investigate the possibility of applying new storm water management techniques and hydrograph runoff models in Canada, this study was oriented in two main directions:

1. Survey of Canadian Design Practices in Urban Drainage Systems - Needs for improved techniques, availability of means for the implementation of new hydrologic methods and awareness of new trends in storm water management, as indicated by a review of the state-of-the-art, were the primary considerations. The survey also included other items of possible interest to municipal engineers.
2. Comparative Assessment of Several Runoff Hydrograph Models - This part of the study is based primarily on measurements of test areas in the U.S.A. and Canada. An evaluation of the Rational Method was also included.

The results presented in this volume are intended for the general information of those working in storm sewer design. Detailed mathematical developments regarding the different models have been avoided and can be found in the references by those interested.

TABLE 1. COMPARISON OF HYDROLOGIC MODELS.

	Model	Surface Routing	Sewer Routing	Quality Routing	Degree of Sophistication of Surface Flow Routing	Degree of Sophistication of Sewer Flow Routing	Accurate Modelling of Surcharging	Flexibility of Modelling of Sewer Components	Explicit Modelling of In-System Storage	Treatment Modelling	Receiving Water Body Model Available	Degree of Calibration/ Verification Required	Simulation Period	Published Documentation	Data Requirements
Models Considered in This Study	EPA-SWMM	Yes	Yes	Yes	High	Moderate	No	High	Yes	Yes	Yes	Moderate	Individual storms	Good	Extensive
	Cincinnati (UCUR)	Yes	Yes	Yes	High	Low	No	Low	No	No	No	Moderate	Individual storms	Fair	Extensive
	RRL	Yes	Yes	No	Moderate	Low-Moderate	No	Low	No	NA	No	Moderate	Individual storms	Good	Moderate
	Unit Pulse	Yes	Yes	No	Low-Moderate	Low-Moderate	No	Low	No	NA	No	Moderate	Individual storms	Fair	Moderate
	Dorsch (HVM)*	Yes	Yes	No	High	High	Yes	High	?	NA	Yes	Moderate	Individual storms or (separate) long term	Poor	Extensive
	MIT*	Yes	No	No	High	NA	NA	NA	NA	NA	No	Moderate	Individual storms	Fair	Moderate
	Rational Method	Peak Flows Only	Peak Flows Only	No	Low	Low	No	Low	No	NA	No	Usually not verified	Individual storms	Good	Low
Other Models	Chicago	Yes	No	No	Moderate	NA	NA	NA	NA	NA	No	Moderate	Individual storms	Fair	Moderate
	Unit Hydrograph	Yes	In combination with surface	No	Low	Low	No	Low	No	NA	No	High	Individual storms	Fair	Moderate
	STORM	Yes	In combination with surface	Yes	Low	Low	No	Low	No	Yes	No	Low	Long term	Good	Low
	Battelle*	Yes	Yes	Yes	Low	Moderate	No	Moderate	No	No	No	Moderate	Individual storms	Poor	Moderate
	WRE-SWMM*	Yes	Yes	Yes	High	High	Yes	High	Yes	Yes	Yes	Moderate	Individual storms	Poor	Extensive
	SOGREAH*	Yes	Yes	?	High	High	Yes	High	?	?	Yes	Moderate	Individual storms	Poor	Extensive
	Hydrocomp*	Yes	Yes	Yes	Moderate	Moderate	No	Low	No	No	Yes	High	Individual storms or long term	Fair	Extensive
	Illinois (ISS)	Yes	Yes	No	Moderate	High	No	Low	No	NA	No	Moderate	Individual storms	Good	Extensive

* proprietary

This study was possible only through the co-operation and assistance of many people. Professor E. Watt from Queen's University independently carried on the measurements at the test area of Calvin Park in Kingston, developed and tested the Unit Pulse Model and participated by stimulative discussions at all phases of the project. Professor Wayne Huber from the University of Florida, one of the authors of the SWMM, kept us informed with the updating of this model and acted as a consultant for its application. Dr. Constantine Papadakis from the University of Michigan assisted in the modification and use of the UCUR model. Mr. Harry Torno from the U.S. Environmental Protection Agency assisted us greatly in our U.S. contacts, provided initial versions of the computer programs and provided many interesting comments with regard to the philosophy of model implementation.

Information and comments from Mr. C.P. Young from the British Road Research Laboratory, Professor H. Preul from the University of Cincinnati, Professor S. Solomon from the University of Waterloo, Professor J. Schaake from M.I.T., Mr. A. Westfall from the Sewer Department of the City of Chicago and many others, were greatly appreciated.

Appreciation is also expressed to the city engineers of the 37 municipalities who participated in the inquiry, to the municipality of North York for providing the assistance for the selection of a test area to Mr. C. Kitchen from the City of Toronto, Mr. J. Brown Commissioner of Works in Scarborough and Mr. B. Ruddy, Commissioner of Works in North York, for the discussion of some of the problems of the study. Mr. J. Boudreau from W. Cosgrove and Associates Montreal, assisted in the collection of information from the municipalities of Quebec.

The research team of James F. MacLaren Limited is particularly grateful to Dr. T. Milne Dick, Head of the Hydraulics Division, Canada Centre for Inland Waters, who acted as the Scientific Authority for the general direction and guidance during the study and to Mr. Jiri Marsalek, who was the main contact with the Scientific Authority, for his direct and active participation in all aspects of this study. Names and contributions of the personnel of James F. MacLaren who participated in this study are acknowledged at the end of this report.

2. STORM DRAINAGE PRACTICE

2.1 Review of New Design Trends

With the increased interest in urban drainage, many new ideas have been suggested for the design and management of storm water drainage facilities. Instead of building ever larger and more expensive channels and conduits for carrying the increased runoff from urban areas, the present trend is to implementing policies and constructing facilities which will reduce the required flow capacity by retarding or reducing runoff. One such method of reducing peak flows from runoff is to provide for controlled ponding.

Since parking lots are a major facet of urban development, it has been suggested that they should be used to provide surface storage during some storms (57). If properly designed, this can be accomplished with little inconvenience to the parking lot users, by sizing the drains to allow runoff from the more frequent minor storms to pass unimpeded to the sewer system. Rooftop storage is also considered a feasible means of peak flow reduction, especially since most existing buildings are already designed to carry the loads which might result from rooftop rainfall ponding.

Several cities have adopted policies which necessitate the use of these types of facilities in urban areas. The Metropolitan Sanitary District of Greater Chicago, for example, limits the amount of runoff allowed from new developments to that which occurred prior to development. In general the ordinance requires developers to construct on-site detention facilities on new residential developments greater than 10 acres and new commercial developments larger than 5 acres (58). In addition to this policy, Chicago has built or financed large underground detention reservoirs to relieve overloaded sewers in areas of older development.

In downtown Denver, Colorado, private developers are required to detain (on-site) the rain falling directly on their properties long enough to allow the upstream flow to pass by, thereby relieving a surcharged condition in the sewer. This criterion has been met by allowing varying degrees of ponding in malls, plazas, parking lots and on rooftops (58).

Another aspect of retention storage being widely considered, particularly in new developments, is known as the blue-green concept. This simply means that artifical ponds and lakes can be incorporated into proposed parkland and green-belt areas to act as retention basins. Since it is generally accepted that the presence of water in or near a development is very desirable from both aesthetic and recreational viewpoints, the additional economic benefits derived from the reduction of peak runoff rates should make them even more attractive to municipalities and developers (57).

One example of ponding for on-site detention is in Earth City, a new planned development in Missouri (57). There, the inclusion of artificial lakes has both reduced the cost of drainage facilities and created an attractive recreational area at the same time.

Examples of planning for storm water retention facilities in Canada are given by several projects being conducted presently by the City of Winnipeg (3, 12). These studies compare traditional sewerage by conventional conduit systems with parkway water corridors and retention lakes. The parkway water corridors incorporate retention within enlarged channels. These channels provide storage and at the same time become the carrying facility in place of the large trunk storm sewers. Published information about the Bunn's Creek Study (12) indicates how elongated retention ponds will be used to enhance recreational use and provide an aesthetic amenity.

The possible overall economies which might result from the utilization of on-site detention schemes can be illustrated by several examples (58). The use of detention ponding on the parking lot of a new trucking facility in St. Louis, Missouri reduced the cost of the required storm sewerage from $150,000 to $115,000. In the 'Earth City' development in the Missouri River bottoms, it was found that a system of finger lakes serving as storm water detention reservoirs would reduce the cost of storm sewers from $5 million to $2 million plus the cost of the 51 acres of land needed for the lakes.

Disadvantages of on-site storm water detention facilities have been found to be related to maintenance and operational problems

(58). If storage is implemented by a municipality, these aspects and legal problems related to the different responsibilities should be carefully analyzed. Factors of importance to the local aspects of detention are the safety hazards of detention ponds, the authority to require on-site detention, the amount of storage required, legal responsibilities for maintenance and operation, the burden of facility cost and the relationships of detention storage to water right laws. Development of specific design criteria, including hydrologic and environmental considerations, are very useful if a new approach to storm water management is considered.

The available literature indicates that many of the storage systems were designed by means of a very simple computation based on mass curves derived from rain fall intensity-duration-frequency curves and a constant runoff co-efficient, or triangular synthetic hydrographs with the peak equal to the flow calculated by the Rational Formula (61). Limitations of the Rational Formula as a design tool will be discussed in Section 3.7. Extension of the use and assumptions of the Rational Formula beyond its initial scope may lead to even greater errors. A more sophisticated technique for storm water management, such as storage, needs a more complex hydrologic study. There is also a definite relationship between the maximum storage utilization, peak flow rate and the characteristics of the inflow hydrograph. Different types of outlets, such as broadcrested weirs, drop inlets, spillways, etc., have to be compared and the difference in the released peak flow might be considered, for example, as indicated in a detailed study of a storage pond (44).

New design criteria also consider another method to reduce urban runoff which is to lower the degree of imperviousness. This is achieved by various means such as requiring roof leaders to drain to lawns, using porous pavements where climate permits, grassed watercourses, gravel-filled channels and seepage pits, and the preservation of natural ravines and green belts. An innovative example of this latter approach is found in the Woodlands development near Houston, Texas (24). In this development, parts of the land with the highest permeability will be left as green areas to enhance ground water recharge and reduce run-off, and natural drainage channels are to be preserved wherever possible.

The pioneering efforts of the conservation authorities and member municipalities in Ontario in this regard are also well recognized.

Environmental considerations such as erosion, sedimentation, pollution of receiving waters from urban storm runoff, and the lowering of ground water tables due to the increased imperviousness of urban areas are other new aspects of urban drainage design. Many of these problems can be alleviated as a by-product of the methods used to reduce peak flows. The draining of roof leaders to pervious areas will increase ground water recharge, and artificial ponds and lakes may act as settling basins to trap excessive sediment content in the runoff, or as holding tanks for the eventual treatment of polluted storm runoff.

Side effects of new practices require careful consideration. One possible effect might, for example, be a higher ground water table which could aggravate infiltration in sanitary sewers.

Another aspect which has been re-evaluated is the approach to the selection of the design frequency. The 'Dual Drainage Concept', applied for the first time in Denver, is the realization that every urban area has two separate and distinct drainage systems. One is the minor system consisting of closed and open conduits and the other is the major system, which is the route followed by flood or runoff waters when the minor system is inadequate. The traditional urban drainage studies considered only the minor system. According to the Denver Design Criteria, both minor and major systems have to be analyzed in a drainage project.

Typical minor systems are usually intended to have sufficient capacity to collect and transport the runoff from a storm that might be expected to occur only once in a 2 to 10 year period. Return periods from 25 to 100 years might be considered in the major system.

Detention ponds which are part of the minor system, however, should be designed with consideration of possible effects of major floods. Major floods are in general simulated by hydrograph runoff methods. If both minor and major systems are studied simultaneously it is desirable to apply the same methodology for the hydrologic simulation, in order to avoid discrepancies in the result.

2.2 Previous Surveys on Design Practice

The National Association of House Builders (NAHB) in the United States in 1972 distributed 250 questionnaires to different local home building associations (5). The 104 responses representing 92 communities in 37 states show the following results pertinent to that country:

1. Design criteria were established by local governments in 76% of the jurisdictions. Improvements in the downstream drainage were the responsibility of the local government in 6 out of 10 areas.
2. Drainage by paved open channels were permitted in 50% of the areas and natural drainageways (sodded channels) only in 36%.
3. Retention basins for temporary storage were encouraged by 42% of the communities. Credit was given to retention basins for reducing the size of the storm sewers in 56% of the communities reporting the use of retention facilities.
4. There is a wide variation in storm design frequency as indicated in the table below:

Storm Design Frequency (yr)	Per cent of Replies
2	3
3	4
5	16
10	29
20	2
25	15
50	11
100	6
Multiple	14

5. The modern dual drainage concept was used in 14% of the areas.

6. The smallest pipe allowed varied from 8" to 29" in diameter while the largest pipe size required before open channel drainage was permitted varied from 24" to twin 72" pipes.
7. Minimum velocities permitted ranges from 2 to 5 feet per second.
8. The following needs for change were indicated:
   a) development of regional plans and standardization of storm water management policies and design standards.
   b) expanded use of detention basins as a means of reducing peak flows.
   c) use of natural drainage facilities such as grassed swales, ditches, and natural stream beds.

The consideration of any of the new storm water management techniques in the design of urban drainage facilities requires a new design tool capable of evaluating alternative solutions. The City of Chicago, for example, was among the first groups to develop a hydrograph model for the simulation of storm runoff from urban areas for the design of storage facilities (74). Many other models have been developed and applied since that time.

A recent inquiry made in the U.S. and information received from EPA indicates that several U.S. consultants are already conversant with the SWMM model, and one of its applications includes the design of a drainage system at a refinery in Montreal. A modified version of the SWMM model is being used by water resources engineers for the redesign of a sewer system in Hamburg, Germany. The HVM model developed by Dorsch Consult was used in Europe in Munich, Darmstadt, etc., and in Canada for relief sewers in Toronto (41). The RRL model has also been widely used in Great Britain for many years (82, 83). A hydrograph model has also been developed recently in Montreal. Hydrograph models are also used in remote control operation of large combined sewers in Minneapolis-St. Paul, Cleveland, Seattle and Detroit, More information on some of these models will be found in Section 3.6.

It seems, however, that the Rational Method continues to be used by most of the consultants and municipalities in the U.S. and Canada.

Damas and Smith conducted a survey in 1966 of 36 municipal authorities in Ontario (59). Pertinent conclusions are indicated below:

1. All 36 municipalities applied the Rational Method.
2. The storm frequency used in sewer design varies widely as indicated in Table 2(a).
3. Inlet times, design velocities and runoff coefficients also vary considerably as indicated in Tables 2(b) - 2(h).
4. About 60% of the municipalities indicated that storm sewer connections were provided to homes.

In 1967, the University of Wisconsin asked 32 cities in the state to design an extension of a hypothetical development with an area of 15 acres using the Rational Method (7). Twenty-three cities prepared designs but only six cities used the method correctly. Of the surveyed cities, 16 did not consider the rainfall intensity as a variable. Two cities using the same frequency and located within 100 miles of each other designed for rainfall intensities that differ by a factor of almost three. Variation in the C coefficient used fro the same soil type was as much as 0.2 to 0.5.

A more recent similar enquiry was conducted by A. R. Pagan in connection with the design of a culvert with a drainage area of about 800 acres (51). He received 28 answers to 57 questionnaires. Again there was a considerable variability in the range of design frequencies used. The time of concentration varied from 52 to 285 minutes. Although there was not enough information given for the proper selection of the C value, answers were given which ranged from 0.10 to 0.50. The design runoff varied from 88 cfs to 705 cfs. Twelve of the respondants were with the New Jersey Department of Transportation and 11 worked with consulting firms.

TABLE 2. RESULTS OF DAMAS AND SMITH INQUIRY [59]

(a) Storm Frequency

Frequency (years)	Residential No.	Residential % of Total	Commercial No.	Commercial % of Total	Culverts No.	Culverts % of Total
1	2	6.1	1	3.1	1	5.3
2	7	21.2	2	6.3	-	-
5	15	45.4	14	43.7	3	15.8
7	2	6.1	1	3.1	-	-
10	7	21.2	11	34.4	4	21.0
20	-	-	1	3.1	1	5.3
25	-	-	2	6.3	9	47.3
25+	-	-	-	-	1	5.3
	33	100.0	32	100.0	19	100.0

(b) Inlet Times

Time (Minutes)	Maximum No.	Maximum % of Total	Minimum No.	Minimum % of Total
5-9	5	18.5	16	55.3
10-14	3	11.1	7	24.1
15-19	9	33.4	4	13.8
20-24	8	29.6	1	3.4
25-29	-	-	-	-
30 & over	2	7.4	1	3.4
	27	100.0	29	100.0

TABLE 2 (Cont'd)

(c) Minimum Design Velocity (Ft/Sec)

Velocity (Ft/Sec)	No.	% of Total
1	1	2.9
2	10	28.5
2.5	14	40.0
3	10	28.6
	35	100.0

(d) Maximum Design Velocity (Ft/Sec)

Velocity (Ft/Sec)	No.	% of Total
0-4	1	3.4
5-9	3	10.3
10-14	17	58.7
15-19	6	20.7
20+	2	6.9
	29	100.0

(e) Coefficient of Runoff (C) - Residential

C	No.	% of Total
0.2-0.29	1	2.8
0.3-0.39	12	33.3
0.4-0.49	16	44.4
0.5-0.59	5	13.9
0.6 and over	2	5.6
	36	100.0

TABLE 2 (Cont'd)

(f) Coefficient of Runoff (C) - Commercial

C	No.	% of Total
0.5-0.59	1	2.9
0.6-0.69	8	22.9
0.7-0.79	9	25.6
0.8-0.89	2	5.7
0.9 and over	15	42.9
	35	100.0

(g) Coefficient of Runoff (C) - Industrial

C	No.	% of Total
0.3-0.39	2	6.7
0.4-0.49	2	6.7
0.5-0.59	4	13.3
0.6-0.69	7	23.2
0.7-0.79	11	36.7
0.8-0.89	2	6.7
0.9-0.99	2	6.7
	30	100.0

(h) Coefficient of Runoff (C) - Schools and Institutions

C	No.	% of Total
0.3-0.39	4	15.4
0.4-0.49	3	11.5
0.5-0.59	5	19.2
0.6-0.69	6	23.2
0.7-0.79	5	19.2
0.8-0.89	2	7.7
0.9-0.99	1	3.8
	26	100.0

2.3 Survey of Canadian Design Practice

2.3.1 Purpose and methodology

In order to assess the current needs, interests and technical resources of Canadian municipalities with respect to drainage design, a detailed questionnaire was distributed to selected cities across the country. The questions ranged from design data and physical criteria for storm sewers to overall drainage policy and attitudes towards new methods and research needs. Special emphasis was placed on the existing problems of flooding and frost conditions. A copy of the questionnaire can be found in Appendix I.

The purpose of the questionnaire was not to obtain data for a statistical breakdown of design practices, but rather to indicate what the major problems are in municipal drainage systems and what the priorities and policies of the municipalities are at present.

A complete tabulation of the results can be found in Appendix II along with a list of participating municipalities.

Out of 66 questionnaires distributed, a total of 37 replies were received. For the purpose of summarizing this information, the cities have been arbitrarily divided into two groups according to population. These are cities greater than 100,000 (17), and cities less than 100,000 (20).

The following is a discussion of the general trends indicated by the replies to each section of the questionnaire.

2.3.2 General information

Receiving Waters (Section 2) - The majority of the cities polled have systems which drain into creeks, rivers and lakes. Halifax, Vancouver, St. John's and Bathurst discharge directly into the ocean.

Activities and Facilities (Section 3) - Only St. John's, Bathurst, Montreal and St. Catharines reported that new storm sewers did not account for a major portion of their design work (related to storm water management). However, about half of the cities indicated that work with old combined sewers and drainage planning is also significant. Only 7 cities reported design work with recent combination sewers (6 of these were in the Province of Quebec).

Most of the cities (twenty-nine) report that runoff computations are done by their own engineering staff or by this staff in cooperation with consultants or developers.

Montreal, Toronto, Hamilton, Winnipeg, Hull and Scarborough do some design work by computer. However, of the other cities not currently using the computer, 15 have facilities available to them, five of which are in-house facilities (Sainte-Foy, Sudbury, Ottawa, St. Catharines and Calgary).

Generally, the records on older systems are reported as being good but 16 cities (43%) report incomplete records. For cities with less than 100,000 population, this figure is 55%. All the cities keep new records on plans while Halifax and Montreal augment this with microfilm records as well. Montreal and Toronto are the only cities which use a data bank at present.

2.3.3 Ponding

Wide variation is evident in both the occurrence and the frequency of ponding that is considered acceptable. Most cities report that Spring and Summer are the seasons in which ponding is most likely to occur. However, St. John's, Guelph, Thunder Bay, Toronto, Scarborough and Sudbury also reported Fall incidences of ponding or water entering basements. St. John's, Bathurst and Kitchener reported Winter ponding and Halifax and Windsor have observed ponding in all seasons.

The City of Winnipeg feels that water rising above the street curbs is acceptable only once in 15 years while many cities will tolerate this situation once a year. Timmins and Fredericton feel that basement flooding once a year is acceptable while Mississauga reports that once in 10 years is the acceptable frequency of occurrence for this condition. Wide variations such as these appear to be the result of local geographic and climatic conditions. The only perceptible regional trend is the occurrence of Winter ponding in the East coast regions.

In general, most municipalities felt that the ponding which occurs is within acceptable limits. Several exceptions to this were reported, however, such as in Hamilton where inlet ponding is observed once in 5 years while once in 15 years is considered acceptable, in Cap-

de-la-Madeleine where manhole covers are reported to pop off twice a year when once in 10 years is considered acceptable, and in Hull where water enters basements twice a year while once in five years is considered acceptable. The smaller cities also appear to have a somewhat greater incidence of serious local flooding (water above curbs) and sewer surcharging (manhole covers popping off) than the larger centres.

Ponding and surcharge lead, of course, to a reduction of flow peaks by creating additional storage. With the exception of Toronto, however, these effects are not explicitly considered in the design.

Twenty-two cities felt that surcharge or surface ponding on a minor system is permissible only once in several years, while Cap-de-la-Madeleine reported several times a year and three others once a year as permissible. All cities except Halifax report that their drainage system is adequate for design conditions, but only 16 cities report that their system has been observed under design conditions. Only six cities allow occasional flooding if the design calls for intermediate pipe sizes; that is, most cities always install the next larger pipe size.

Maintenance Problems (Section 5) - The major problems reported from the larger centres vary from city to city. Halifax, Ottawa and Thunder Bay consider general cleaning of grates and inlets to be the biggest problem. Kitchener and Winnipeg report the collapse of old sewers and the rebuilding of catch basins. Ottawa, Windsor and Hamilton consider flooding of intersections and catch basins to be the major problem. Freezing is important in Ottawa and Calgary and roots are considered a major problem only in Winnipeg.

Of the 20 cities smaller than 100,000 population, the general cleaning of grates, inlets and catch basins is the biggest problem for nine of them. Four cities report that freezing is the biggest problem and three report that it is the flooding of intersections and basements. Problems reported with roots (Sherbrooke, Cap-de-la-Madeleine); collapse of storm sewers (Anjou); and cleaning of storm sewers (Timmins) are not frequent of major concerns. Generally, the problems of children and animals and the cleaning of sewers are considered normal or infrequent.

2.3.4 Construction and maintenance

Materials used for Storm Sewer Construction (Section 6) - All cities report that concrete pipe is used for the majority of storm sewers constructed. However, 19 cities occasionally use corrugated steel pipe (up to 15% in North Bay) and 8 use some asbestos cement pipe (up to 35% in St. Laurent). In addition, Hamilton and Montreal use some brick; Toronto, Regina and Quebec City use some clay pipe. Toronto, Montreal and Vancouver have also cast concrete sewers in place. The city of Sherbrooke is currently planning to introduce the use of plastic pipe as well.

Inlets and Gutters (Section 7) - Only 13 cities reoort using depressed inlets with the average inlet depression ranging from 1/2" to 4". The average inlet length varies from 1.25 to 2.50 feet and the average inlet spacing varies from 150 to 500 feet with about 300 feet being the most common. Three cities may have misinterpreted the question about inlet length, however, having reported values of 30, 50 and 400 feet.

Method of Snow Disposal (Section 8) - Land disposal of snow is the most common method used. In addition, Montreal and Quebec City use snow melters and Kitchener reports using sanitary sewers for some snow disposal.

Snow is dumped directly into receiving waters (mostly rivers) by 13 municipalitites while Toronto and Windsor report this practice only in emergencies after heavy snowfalls. In addition, Guelph and Scarborough report that land disposal sites are used which drain directly to storm sewers or watercourses.

Problems Associated with Freezing and Cold Weather (Section 9) The major problem reported is the freezing of catch basins and culverts and the resulting flooding during melting or rainstorms. The cost estimates vary widely depending on the size of the municipality and local climatic conditions and they are not directly comparable since the costs associated with the various snow and ice problems were not individually defined. The annual costs for the smaller cities varied from $4,000 in Bathurst to $100,000 in Pierrefonds and Sudbury. For the larger centres, the annual costs ranged from $5 - $6,000 in Mississauga to $700,000 in Halifax.

2.3.5 <u>Engineering criteria for design of stormwater drainage facilities</u>

<u>Standards and Design Criteria (Section 10)</u> - Only Laval, Winnipeg and Sherbrooke have not as yet specified the adoption of design criteria. However, Winnipeg, Halifax, St. Catharines, Fredericton, Valleyfield, Guelph, Sarnia and Calgary are developing new criteria.

Only three cities, Chicoutimi, Valleyfield and Pierrefonds require that the runoff rate from an area where drainage improvements are planned be no greater than that of the area in its natural state. Of the larger cities, only Montreal and Ottawa specify that the runoff rate should not exceed a particular fraction of the rainfall, while 9 of the smaller cities do so. Only 7 cities require on-site detention in their criteria. These are Chicoutimi, Pierrefonds, Guelph, Halifax, St. Catharines, Winnipeg and Calgary.

All except 3 of the larger cities and 9 of those less than 100,000 population require that their criteria be satisfied in order to secure building permits.

It is interesting to note that, in general, the smaller cities have shown more interest in reducing the volume of storm water runoff than have the larger ones, particularly in the adoption of a policy to maintain natural runoff rates and in specifying maximum runoff rates in their design criteria.

<u>Standards (Section 11)</u> - A total of 11 cities design according to the ASCE Manual while 18 others use specific standards and Calgary and Guelph use both.

<u>Methods for Runoff Computation (Section 12)</u> - All the cities use the Rational Method, except Sainte-Foy which uses the McMath formula, with 32 doing the computation by hand and 5 by hand and computer (Montreal, Scarborough, Toronto, Hamilton and Winnipeg). In addition, Montreal, Toronto and Winnipeg use hydrograph methods.

<u>Design Storm Frequency (Section 13)</u> - For residential, commercial and industrial areas and for driveway culverts, the design storm frequency varies from 2 to 25 years with 5 years being the most common. Sainte-Foy reports using a 25-year frequency, Montreal and Scarborough 10, and Thunder Bay and Kingston 2 years. For major road culverts the

design storm frequency is highly variable from city to city, ranging from 2 years in Thunder Bay and Kingston to 50 years in Winnipeg, Manitoba and Waterloo, Ontario.

This wide range of design frequencies may not be as significant for the pipe sizing as it seems since the choice of runoff coefficients is also very important. For example, the choice of a large design storm might be compensated for by the selection of a small runoff coefficient. (This may be the case in Guelph or Ottawa for instance.) From the replies on the choice of runoff coefficients, however, it appears that those municipalities using a more conservative return period often used a conservative runoff coeffieicent as well.

The City of Sainte-Foy, for example, uses 25-year return period for the design of all culverts and runoff coefficients from 0.5 for residential areas to 0.9 for downtown commercial districts, all of these values being in the upper range of those used by other cities. Of course, the economic cost of flooding, which varies greatly from city to city, is also an important consideration.

Characteristic Storm Data (Section 18) - Most cities use characteristic intensity-duration-frequency curves applicable to their region. The highest intensities are used by the City of Toronto and the lowest by Vancouver. Eight of the small municipalities (6 of them in Quebec) use only one rainfall intensity, corresponding to a single return period and duration. This may reflect a lack of precipitation data from which curves could be developed.

Application of the Rational Method (Section 17) - The maximum inlet times reported varied from 40 minutes in St. John's to 8 minutes in Toronto. The minimum inlet times ranged from 20 minutes in Valleyfield and Windsor (which also reported 20 minutes as the maximum inlet time) to 5 minutes in 12 different cities.

Runoff coefficients also vary widely from city to city as indicated below:

	Minimum		Maximum
Parks	.05 (Ottawa)	-	.35 (Guelph)
Residential	.20 (Montreal	-	.80 (Ottawa, Pierrefonds)

Industrial	.30 (Brantford, Calgary)	-	.90 (Guelph, Winnipeg)
Commercial	.36 (Vancouver)	-	.95 (Guelph)

It is evident that the runoff coefficients in the Rational Method cannot be exactly specified or standardized as they vary widely from case to case and are also dependent somewhat on the engineer's experience and judgment.

Design Velocities (Section 17) - Little response was received concerning the questions about maximum and minimum design velocities in channels with only 11 cities replying, but most cities did reply to the same questions for pipes. The results are tabulated below:

	Range of Max. Design Velocity (fps)	Range of Min. Design Velocity (fps)
Earth Channel	3 (Several) - 6 (Guelph)	1 (Hull) - 3 (Calgary)
Lined Channel	4 (Hull) - 25 (Valleyfield)	2 (Several) - 10 (Valleyfield)
Pipes	7 (Sarnia) - 25 (Sainte-Foy)	2 - 3

In general, it appears that there is some consistency in the maximum and minimum design velocities used for pipe flow with the maximum value in the range of 10-15 fps and the minimum ranging from 2-3 fps. The few extreme values may be the result of special local conditions.

Master Drainage Design (Section 14) - Seven cities (six of them less than 100,000 population) report that major and minor drainage systems are not clearly defined. None of the smaller centres used less than 1/50 years while only three of the larger ones did. Fifteen of the cities compare open channel and closed conduit alternatives and all do selection on an economic basis. However, it is felt that this question was misunderstood by many municipalities. The questions concerning selection on an economic basis and other criteria for selection were meant to refer to the comparison of open channel and closed conduit alternatives. Since several cities reported selection on an economic basis, even though they stated that open channel and closed conduit alternatives were not compared, it is evident that the intention of the questions was not clear.

Other criteria mentioned were aesthetics, maintenance, conservation of natural water courses and plans to separate storm and sanitary sewers.

The reported sewer costs varied widely depending on local conditions and pipe size, but many municipalities did not have figures available. In the smaller centres, the costs ranged from $10/ft in St. John's and Waterloo to $60/ft in Valleyfield. (Pierrefonds reports a total cost of $4 million for all residential storm sewerage.) In cities with a population greater than 100,000, the costs range from $8/ft for the main sewer plus $255/home for weeping tile storm sewer connections in Thunder Bay to $50/ft for larger pipe sizes in Quebec City. Toronto reported a net cost of $4,390/acre and Winnipeg $4,000/acre.

Detention (Section 15) - A large number of cities (fifteen) report that detention should be employed to reduce design peak flow, including 47% of the cities with more than 100,000 people. However, only 8 municipalities presently consider detention in design. Facilities presently in use to increase detention include roof tops in Mississauga and Halifax, streets in Chicoutimi, sewer pipes in Valleyfield, ponds in Winnipeg, and parking lots in Halifax and the facilities are evenly divided between private and public ownership.

Storm Sewer Use Regulations (Section 16) - Only Brantford, Sainte-Foy, Thunder Bay and Regina presently require that roof drains discharge onto property rather than directly into the storm sewers. All other municipalities allow connections to storm sewers. Halifax also allows roof connections to combined sewers while Toronto requires that no connections be made to sanitary sewers. Sudbury allows connections to sanitary sewers where storm sewers are not available. No other cities require that roof drains be connected to the storm sewer system only. In general, there has been little change in these policies with only 4 cities reporting changes over the last 25 years.

Foundation drains may be connected to storm sewers in all cities except Laval, Winnipeg, and Sainte-Foy. Eleven centres allow foundation connections to sanitary sewers, of which Sainte-Foy and Sudbury also do not allow connections to the storm sewers. While St. Catharines and Waterloo, Ontario prefer to use storm sewers for foundation

drains, few existing storm sewers in these cities are deep enough for this purpose. Therefore, in these cities, sanitary sewer connections are allowed. There also appears to be a trend away from the use of sanitary sewer connection to storm and combined sewer connections with six municipalities reporting this change over the last 25 years.

Catch basin spacing varies from 100 ft in Valleyfield to 1,000 ft in Calgary with most replies being in the 200 - 300 ft range. Minimum sewer size also varied widely from 6" reported in Quebec City to 15" in Anjou and Valleyfield, while most cities reported a minimum sewer size of 10-12". Minimum sewer cover varies from 0.75 ft in Hull to 9 ft in Montreal and Hamilton. The sewer distance from the centre line of the road varied from 0 ft in seven cities up to 43 ft for some sewers in Sherbrooke. The maximum distance to the first inlet also varied widely from 15 ft in Pierrefonds to 6,000 ft in Hull.

Thirteen cities report that inlets are designed using the ASCE Manual No. 9. Fifteen did not reply to this question, perhaps indicating that no inlet design calculations are done. Five hundred and fifty feet was given as the maximum manhole spacing in Calgary whereas 250 feet was reported as the maximum spacing in Valleyfield and Hull. Of all cities replying, only Bathurst does not propose to install catch basins in new developments, while 19 cities will not allow double house connections. Finally, only 19 municipalities out of 36 consider bend and junction losses in sewer design.

Problems of Concern and Research Needs (Section 20) - The majority of the cities expressed some interest in all of the problems. Research into pollution is considered a high priority by eight cities while 23 others expressed interest in it.

Of the 15 cities that felt detention was required (Section 15) only six (Sainte-Foy, Hull, North Bay, St. Catharines, Windsor, Winnipeg) consider research into this area to be a high priority.

Interest was expressed by 25 cities in new runoff models, but none felt that this was of high priority. There is some contradiction in these results, however, because the introduction of design techniques such as optimization and detention and the minimizing of pollution requires that new models capable of handling these problems be perfected first.

It is also worthy of note that 6 cities considered improved land use a high priority (Valleyfield, Hull, North Bay, Sarnia, Halifax, Winnipeg). At the same time, however, 8 cities felt there is no need for this.

The following is a frequency tabulation of the response to these questions:

RESEARCH AREA / Priority	New Runoff Models	Optimization Techniques	Improved Land Use	Pollution for Urban Runoff	Detention & Storage	Design Storms	Urban Erosion
High Priority	0	3	6	8	6	3	3
Interested	25	21	13	23	19	22	17
No need	5	3	8	0	5	4	7
Totals	30	27	27	31	30	29	27

2.3.6 Northern conditions

Two American cities, Fairbanks and Anchorage, Alaska, have replied to the questionnaire and their responses are summarized in the following sections. Some information has also been made available by the government of the Yukon Territory. The following discussion summarizes the major points of interest concerning storm sewers in northern urban areas.

Yukon Territory

The Rational Method has been used to design a storm drainage system in Whitehorse. Elsewhere in the Territory, however, a pragmatic design approach is used because the runoff coefficient has not been accurately determined for most areas due to the great variation of soil, plant and frost conditions.

Most communities rely on ditches with either steel or corrugated iron pipe culverts for drainage. The main problems result from the plugging of culverts by freezing melt waters or sediments which tend to settle out in the culverts leading to flooded and icy roadways. Usually, frozen culverts are steamed out but a few locations use elec-

trical resistance type heating cables in the culvert. Because of the success of this latter procedure, the Territory intends to increase its use where possible. Where communities are built in areas of discontinuous permafrost, surface ditching is generally used for drainage since drainage mains are unreliable because of frost heaving.

Fairbanks, Alaska (Pop. 18,000)

Spring runoff is a problem but the freezing of shallow storm drainage lines is more serious. There is currently some discussion towards the use of combined sewers since the annual precipitation is only 11 inches and the resulting high proportion of sanitary flow might keep the sewers unfrozen during the critical periods of Spring runoff.

The present trend is towards using reinforced plastic pipe rather than the existing wood stave pipe which is becoming expensive. However, the wood stave pipe is very good at withstanding freeze-thaw cycles and steam shock when thawing the lines in Spring. The estimated cost for providing storm sewers is about $3,100 per acre.

Anchorage, Alaska (Pop. 75,000)

The Rational Method is used in storm sewer design and a computer will be used for calculations in the near future. The runoff coefficients reported are comparable to those used in southern cities.

Corrugated steel pipe is the main construction material and the estimated cost for sewer construction is $2,500 per acre.

Freezing is the biggest problem and non-frost susceptible gravel is required around storm sewer lines which consist of about 75% corrugated steel and 25% concrete pipe. Also, sanitary sewers and water lines must be insulated when storm sewers pass in close proximity.

2.4 Conclusions on Drainage Practice

1. Urban drainage practice in North America is in a transition stage. Although traditional methods continue to be applied on a large scale, there are also many attempts to apply new concepts such as different types of storage, runoff limitation, a uniform analysis of major and minor drainage systems, etc. New simulation methods are also increasingly applied instead of the Rational Method.

2. The most frequent problems of Canadian municipalities are associated with flooding. However, few cities report serious difficulties in this regard. In general, the smaller cities appear to have more problems with flooding than the large centres.

3. At present, the Rational Method continues to be nearly universally applied to storm drainage design in Canada whether for northern or southern conditions. Wide discrepancies in parameters from city to city reflect differences in local conditions, but they also illustrate the degree of subjectivity associated with this method. It is difficult to estimate which designs are more conservative, since a large design storm may be used in conjunction with a small runoff coefficient.

4. Although most of the Canadian cities have design criteria, only 50% use standards specific to their own conditions. Standards are in general, oriented toward traditional drainage practices.

5. Twenty-one out of thirty-seven cities have in-house, or access to, computer facilities. However, computers are used for storm sewer design in very few cities at present.

6. Only 8 of 37 municipalities do consider detention in storm sewer design. Fifteen cities, however, feel that research on detention is required while only 6 consider that this is an area of high priority. Other management problems often considered to be of high research priority are pollution (eight), and land use planning (six).

7. Canadian municipalities have a widespread interest in new urban storm water management techniques and new computer models. However, it appears that more information regarding the interrelation between new hydrologic modelling techniques and new management techniques is needed.

3. ASSESSMENT OF URBAN HYDROGRAPH RUNOFF MODELS

3.1 Preliminary Considerations on Hydrograph Runoff Models

3.1.1 Description of phenomena and basic concepts

An urban watershed contains two different types of elements: collecting channels (gutters, lateral, main and trunk sewers) and surfaces, which may be impervious with direct connections to the sewer system, or pervious. Pervious areas may also contain scattered disconnected impervious areas. The degree of imperviousness, which is the ratio of the directly connected impervious area to the total watershed area, is a parameter defining the overall influence of urbanization.

During the precipitation on a pervious area, water is continuously being abstracted by infiltration. Other abstractions considered in rural hydrology, such as interception and evaporation are usually negligible for urban storm events. If the rainfall intensity exceeds the possible rate of infiltration, natural depressions collect some of the excess precipitation creating the depression storage. Since the depressions have different depths and filling rates, overland flow commences as soon as the infiltration capacity is satisfied.

The depth of water detained in the overland sheet flow forms the detention storage on the pervious area which, combined with the depression storage, forms the total surface storage. Paved areas are considered impervious and therefore have negligible infiltration losses. However, the depression and detention storages, although much smaller than for the pervious areas, cannot be considered negligible.

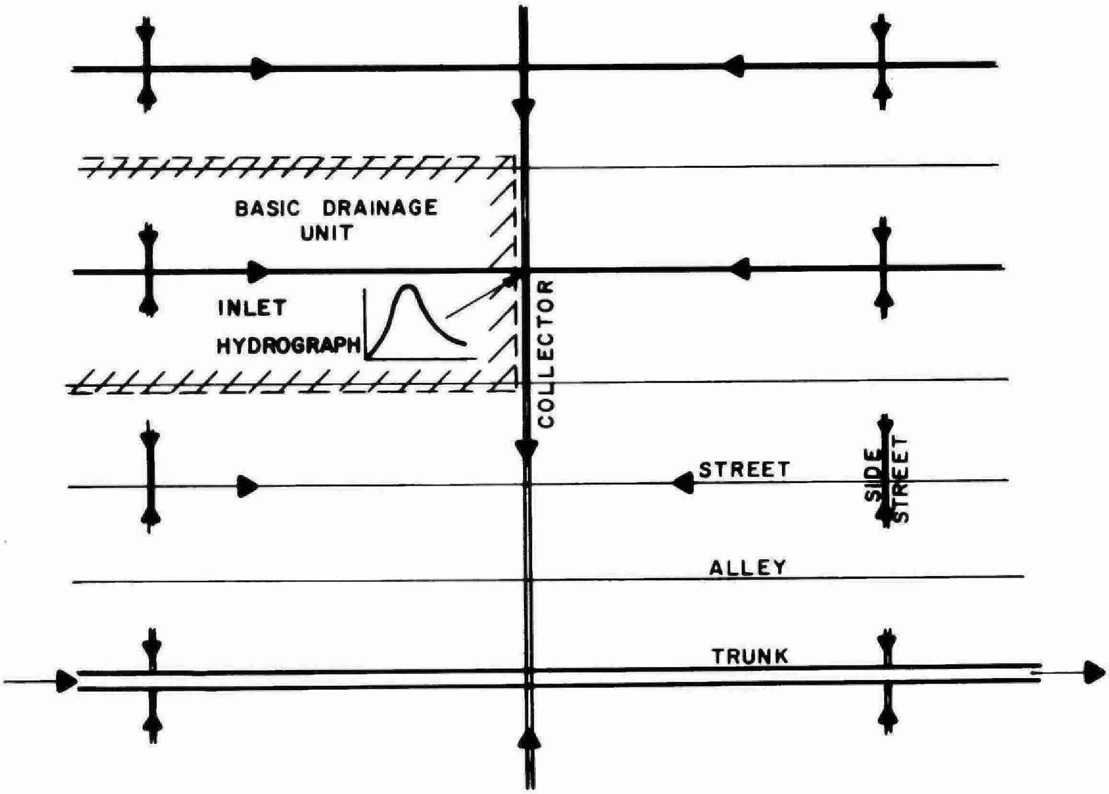
Overland flow from both types of areas is collected in gutters or ditches and subsequently enters the sewer system. The estimation of this overland supply by routing methods is theoretically possible for a plane surface. Prediction for real situations is complicated, however, because of the non-uniformity of the areas involved and the difficulty in estimating the parameters for the various losses. The routing of the supply hydrographs through the collecting system is a hydraulic problem which can be handled by proven methods. Several routing methods are available with different degrees of sophistication

(see Appendix IV). It is not necessary to always apply the same routing method for all the elements of the network. Storage effects in the small lateral sewers are less significant than in the main sewer system. At the same time, because one increment in pipe size greatly increases flow capacity in small pipes, the exact determination of peak runoff is less critical for the smallest sewer laterals. If different routing techniques are applied, the sewer district has to be analyzed first and basic subcatchments defined (see Figure 2). The generated hydrograph from each of these basic subcatchments is input into the main sewer system. Only these outlet hydrographs of the basic subcatchments are routed by more sophisticated methods.

The degree of schematization used in the definition of the subcatchments, and the selection of the pipes to be modelled requires some experience. The schematization is also important, as the cost of collecting input data describing the catchment characteristics is proportionate to the degree of detail considered in the definition of subcatchments.

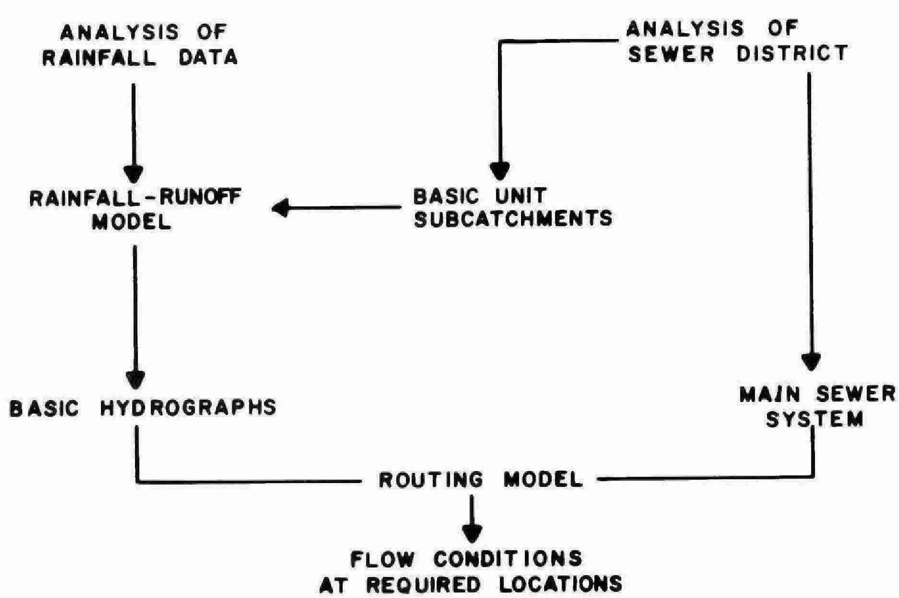
For the purpose of this study, however, it was considered that if acceptable results are obtained for a detailed schematization, the corresponding hydrographs may be used for future calibration of more simplified representations of the systems. Any user who becomes familiar with a given model can determine the sensitivity of the model to varying degrees of schematization by a few test runs.

Assuming an accurate routing technique is used in problems involving large areas, the major sources of error lie in the simulation of outlet hydrographs from the basic subcatchments. On the other hand, runoff models are usually assessed by the comparison of computed and measured hydrographs at the outfall of test areas. If a test area is divided into several basic subcatchments, it is difficult to determine the source of these errors. It was felt, therefore, that the study should concentrate on the modelling of the basic subcatchments. The size of these subcatchments in previous studies varies from a few acres to several hundred acres. It is considered, however, that the selection of subcatchments with less than 10 acres may increase unnecessarily the cost of



EXAMPLE OF A SEWERED DISTRICT
AND A BASIC SUBCATCHMENT

Fig. 2



ELEMENTS OF THE SIMULATION
OF URBAN RUNOFF

Fig. 3

computation. On the other hand, subcatchments with more than 100 acres may lead to errors due to pipe routing, since the error introduced by an approximate routing method increases with the length of the conduit (see Section 3.4.3). The model testing was therefore conducted mainly on areas between 10 and 100 acres in size. Additional information on large areas will also be available in the near future from studies carried on independently in the U.S.A. and Australia (30).

3.1.2 Model calibration and validation

Models used for the simulation of urban runoff have, as an input, a real or fictitious design rainfall and, as an output, the runoff at one or several points of the sewer system.

Although some models include consideration of all of the systems involved in the runoff process and the corresponding physical phenomena, an exact representation of nature is not the purpose of the model builder. By the proper selection of equivalent parameters and simplifying assumptions, an attempt is made to create a tool which gives a useful result with a reasonable balance between cost and accuracy.

Natural hydrologic systems are so complex that any model must neglect at least some of their aspects. The modelling problem, therefore, is to select the most important aspects to be represented and to decide how these should be simulated. Because of this, models have different characteristics and can be classified as linear or non-linear, analytic or synthetic, lumped or distributed, black box or imitating the physical system and so on (66). The availability of input parameters and the cost of data collection are also important considerations.

Runoff models used in urban hydrology are deterministic in general because measurements over a long period of time for statistical analysis of peak flows are not available. Even if a long period record is available, changes in the degree of urbanization and other modifications to the watershed may make statistical analysis inapplicable. Lengthy rainfall records are usually available, however, and rainfall analysis is therefore an important part of the urban drainage study process, as shown in Figure 3.

The return period of a peak flow is not the same as the return period of its associated rainfall event. This results from the influence of antecedent moisture conditions on the runoff generated by a particular rainfall. The determination of the return periods of various recorded peak flows therefore requires the long-term simulation of a series of rainfall events considering the antecedent moisture conditions in each case (65). Most drainage studies, however, use only one event simulation models with a somewhat arbitrary assumption concerning the influence of antecedent precipitation.

Any model has parameters which may or may not have a physical counterpart. Some of the parameters can be determined from physical data which can be measured or assessed by the user without difficulty, such as the degree of imperviousness or the roughness coefficient of a pipe or a surface. Most models, however, involve parameters which are not readily determined, such as the depth of depression and storage and infiltration capacity characteristics. The irregular shape of the watershed is replaced in some models by equivalent areas with a uniform slope and, in this case, the equivalent slope has to be estimated. Because of the indeterminate nature of these factors, the model builders have to calibrate the values of the parameters by fitting the simulated hydrographs to some of the available measurements. At the present time this can be done more economically for this type of application by trial and error than by computerized optimization techniques. If an acceptable goodness of fit is achieved, the model is then validated by comparing other measured samples with the computed output. The fact that a model compares favourably with measurements for a limited number of events may be just the result of the calibration process, however, and it is not necessarily true that similar results will be obtained for other input data.

Once a model is calibrated and validated for real rainfall-runoff data, it may be used in design. The return periods of design storms are usually selected on the basis of economic criteria. These design storms are, in general, more severe than those used in calibration and validation of the models, since records extending beyond a 5 or 10 year period (typical return periods for design storms) are rare.

However, it is assumed that if a model satisfactorily predicts runoff hydrographs for test cases, it will also give a reasonable prediction for design conditions.

3.1.3 Design storms

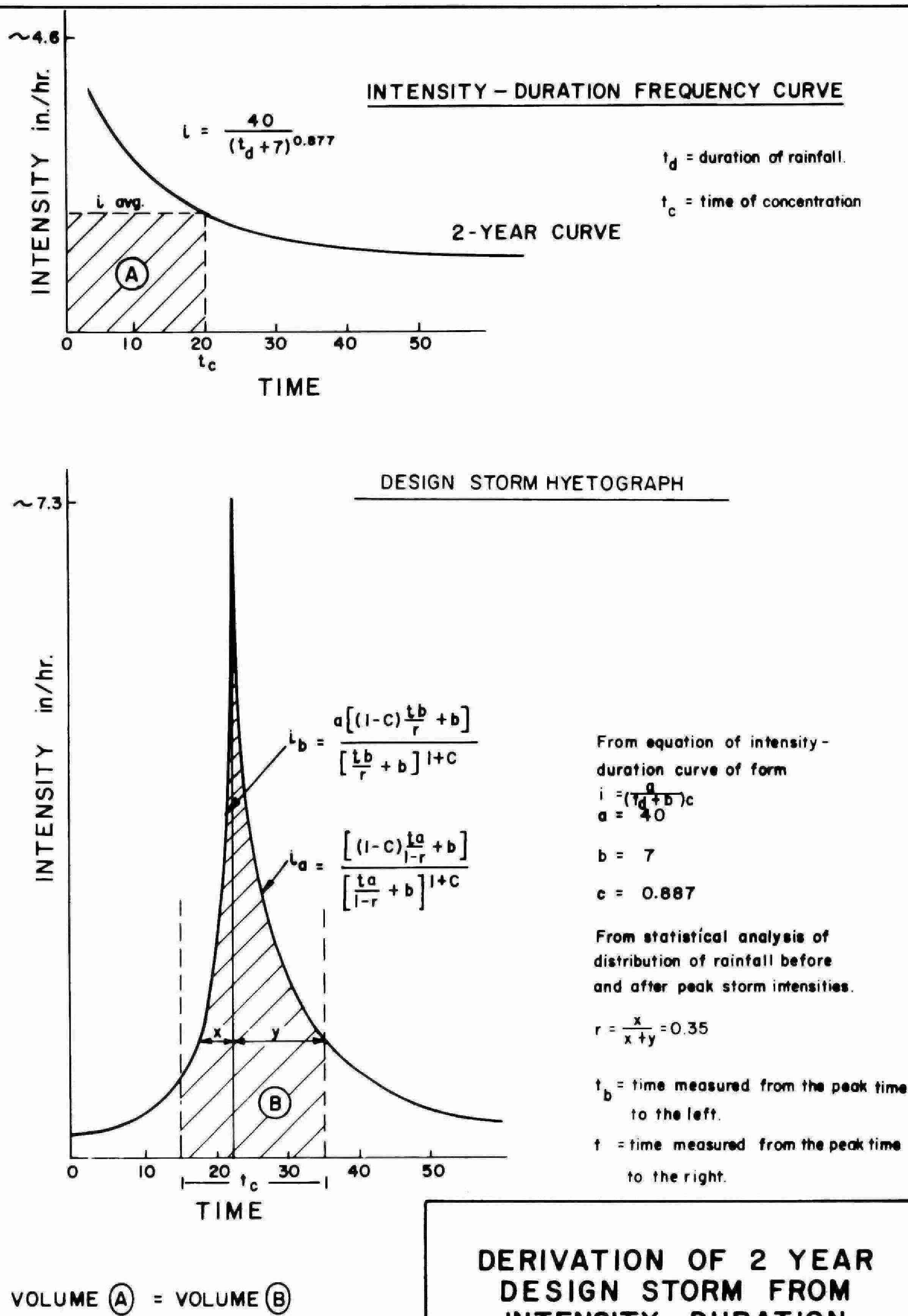
The development of a design storm (Figure 4) is a part of the sewer analysis process and although it is not extensively discussed in this report, it is understood that development of the design hyetograph and rainfall-runoff simulation are of equal importance for the model user.

For the Rational Formula, the design storm is a hypothetical rainfall of uniform intensity. In current practice, design storm hyetographs are developed for use with runoff models which meet the following conditions:

1) For any given duration, the maximum average intensity equals that of the intensity-duration curve rainfall of the same frequency.
2) Characteristic parameters of the hyetograph, such as the total duration of the rainfall and the volume distribution before and after the peak intensity, are obtained by statistical analysis of many rainfall events.

The most frequently used method for the derivation of a design hyetograph is that developed in Chicago by Chu and Kiefer (39), which is presented briefly in Figure 4. There is obviously a considerable difference between the nature of the rainfall input in the Rational Method and the runoff hydrograph methods.

For hydrograph methods, the same rainfall is considered for all points in the system. For a small subcatchment, the critical part of the storm occurs, of course, during the short period of very high intensities while the initial rainfall preceding this period creates wet antecedent conditions. Antecedent conditions can also be estimated by the statistical analysis of rainfall events. In some regions, for instance, a Summer design rainfall with a higher intensity, shorter duration and dry antecedent conditions may be less critical than a Spring or Autumn storm with lower intensity, but with high soil moisture content or frozen soil conditions.



DERIVATION OF 2 YEAR DESIGN STORM FROM INTENSITY - DURATION FREQUENCY CURVE

FIG. 4

3.2 Selection of Runoff Hydrograph Models and Previous Model Comparisons

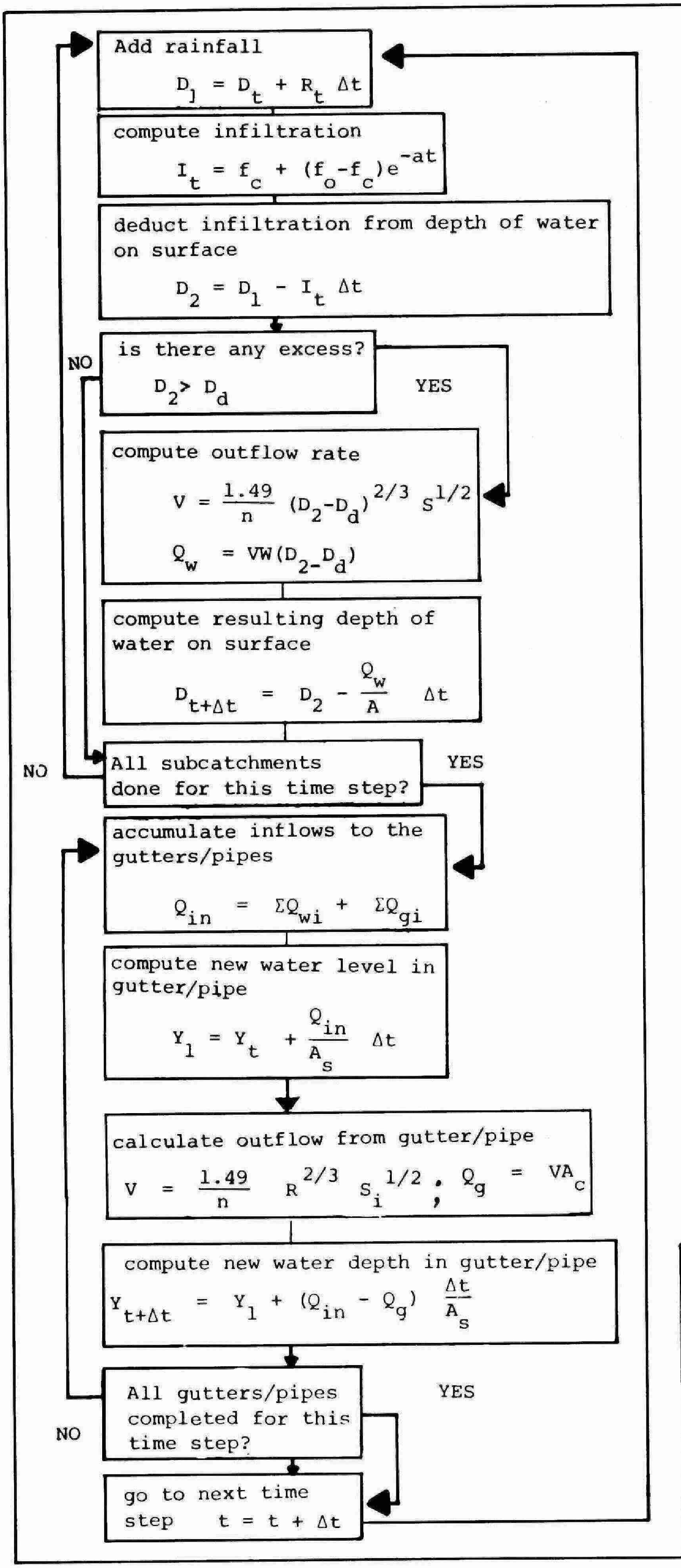
3.2.1 Brief presentation of selected hydrograph models for this study

There is, at present, a large number of runoff hydrograph models which is indicative of the general interest in this approach to urban drainage problems. Some of these models are proprietary, in which case, even if the basic assumptions are known, the program is not generally available. Other models have only fair documentation, making their application difficult. Some models consider runoff simulation only for the analysis of overflows, quality simulation or remote control operation of large sewer systems (see Table 1). The best documented non-proprietary models suitable for the design and analysis of storm sewers at the time when this study was initiated were the following:

1) The EPA Storm Water Management Model (SWMM) developed by a consortium formed by Water Resources Engineers, Metcalf and Eddy and the University of Florida (47).
2) The model developed by the University of Cincinnati (UCUR) (56).
3) The Transport and Road Research Laboratory Model (RRL) developed and extensively used in Britain (82).

Each of these three models can be combined with routines simulating the sanitary flows and used for combined sewers. The SWMM is actually designed for this purpose and the runoff routine considered in this study is only a part of a complex model for the study of pollution abatement for combined sewers. These aspects, however, are beyond the terms of reference of this study which is oriented primarily toward the design and analysis of drainage systems. The SWMM, UCUR and RRL models were initially recommended by the Scientific Authority sponsoring the study.

The first two models indicated above simulate the overland flow and the routing in the collecting pipes and gutters for each time step by mathematical formulations, given in Figures 5 and 6, which are based on the different parts of the physical process described in Subsection 3.1.1. The physical system is represented by a number of



D_1 = water depth after rainfall
D_t = water depth on the surface at time t
R_t = rainfall intensity in time interval Δt
I_t = infiltration rate at time t
f_o, f_c, a = coefficients of Horton's equation
D_2 = water depth after infiltration

D_d = specified retention depth
V = velocity,
n = Manning's coefficient
S = ground slope
Q_w = total outflow rate
W = subcatchment width

A = surface area of the subcatchment

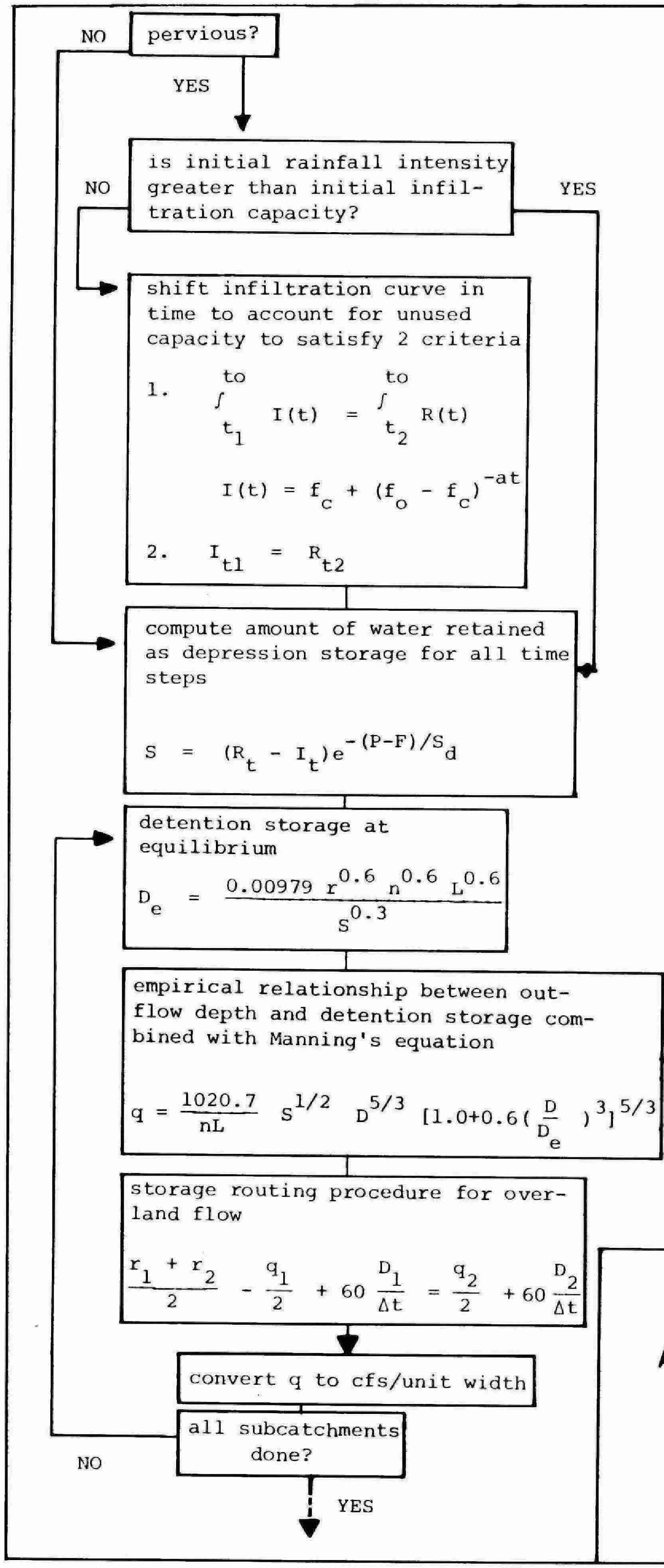
Q_{in} = inflow to a gutter
ΣQ_{wi} = outflow from tributary subcatchments
ΣQ_{gi} = flow rate of upstream gutters

Y_1 = new gutter water depth
Y_t = existing water depth in the gutter
A_s = mean water surface area between Y_1 and Y_t

R = hydraulic radius
S_i = the invert slope
Q_g = outflow from gutter
A_c = cross sectional area at Y_1

ALGORITHM OF THE SWMM MODEL

Fig. 5



I_1 = initial infiltration capacity

R_1 = initial rainfall intensity

I = Horton's infiltration function

R = rainfall hyetograph

t_o = start time

I = infiltration rate

f_o = initial infiltration rate

f_c = minimum infiltration rate

a = a constant

t = time

S_d = total depression storage

P = total volume precipitated

F = accumulated volume of infiltration

S = depression storage supply

D_e = detention storage at equilibrium

r = overland flow supply rate

S = surface slope

L = length of flow

n = Manning's roughness coefficient

q = runoff/unit area

D = current detention storage

ALGORITHM OF THE UCUR MODEL

Fig. 6

accumulate flows in the triangular curb gutters

$$Q = q_1 L + Q_o$$

Q = total gutter flow

L = length of gutter (= width of sub-catchment)

Q_o = upstream inflow

↓

calculate average velocity of inflow

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

$$V_{avg} = \frac{(V_1 Q_1 \Delta t + V_2 Q_2 \Delta t + \dots + V_N Q_N \Delta t}{Q_1 \Delta t + Q_2 \Delta t + \dots + Q_N \Delta t}$$

V, etc. = velocities with which volumes $Q_1 \Delta t$, etc. travel downstream

↓

offset inflow hydrograph by travel time

$$T_{OFF} = \frac{L_p}{V_{avg}}$$

T_{OFF} = time offset

L_p = conduit length

↓

add offset hydrograph and new gutter input

↓

all conduits done?

NO → calculate average velocity of inflow

YES ↓

OUTLET HYDROGRAPH

ALGORITHM OF THE UCUR MODEL (continued)

Fig. 6 (cont.)

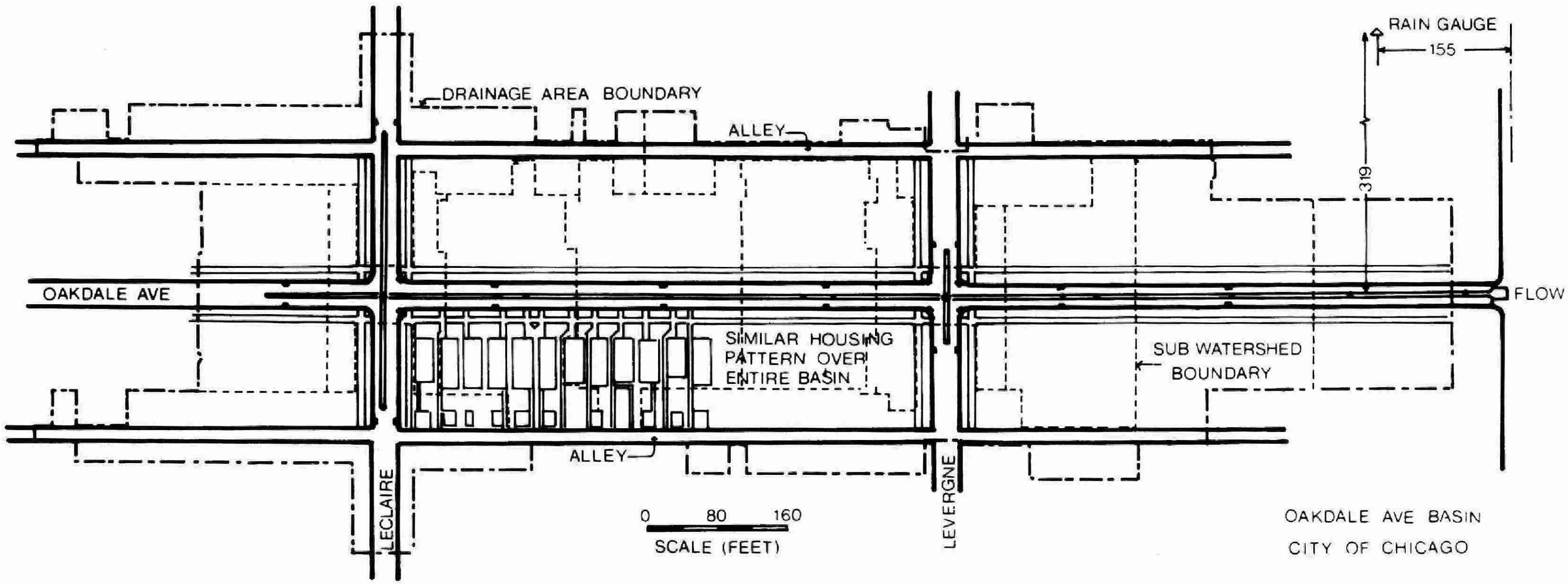
equivalent rectangular areas (compare Figure 7 and Figure 8) which may be pervious or impervious and have different characteristics for the slope, depression, storage, roughness, etc.

The principal assumptions of the RRL model are presented in Figure 9. This model is much simpler than the previous ones as it considers only the contribution of the impervious area, and neglects the detention storage. It also uses the inlet time concept of the Rational Formula. The model considers conduit storage, however, and can take into account the real hyetograph.

With the approval of the Scientific Authority, other models were analyzed for inclusion in this study. The non-proprietary model, NERO, developed and extensively used by the Municipality of Chicago was found quite similar in approach to the Cincinnati model and was therefore not considered further (38).

Another type of model extensively used for hydrologic studies in Canada is the unit hydrograph. This is a linear and lumped model which does not attempt to give the flow conditions in each point of the system. Its initial form is of the black box type. Several attempts have been made to apply it also for urban conditions (27). One of the most recent and successful variations is the unit pulse approach based on the linear reservoir concept described in Figure 10. Being a lumped model which combines the hydrologic characteristics of an area into one parameter, the model is applicable to the modelling of very large areas without detailed definition. At the same time this model has not yet been used for municipal design purposes. It was recommended, however, for studies of urbanization effects (27), and for operation of large sewer systems. A model presently studied at Queen's University in Kingston (QUURM) uses the Unit Pulse concept for small subcatchments combined with a routing routine. Because of its design orientation, the assessment of the QUURM was included in this study.

Towards the end of the study, it was possible through the cooperation of Dorsch Consult of Toronto to include also a partial comparison of the Dorsch proprietary model with the other models in the report. The Dorsch Model is of the same type as UCUR although the

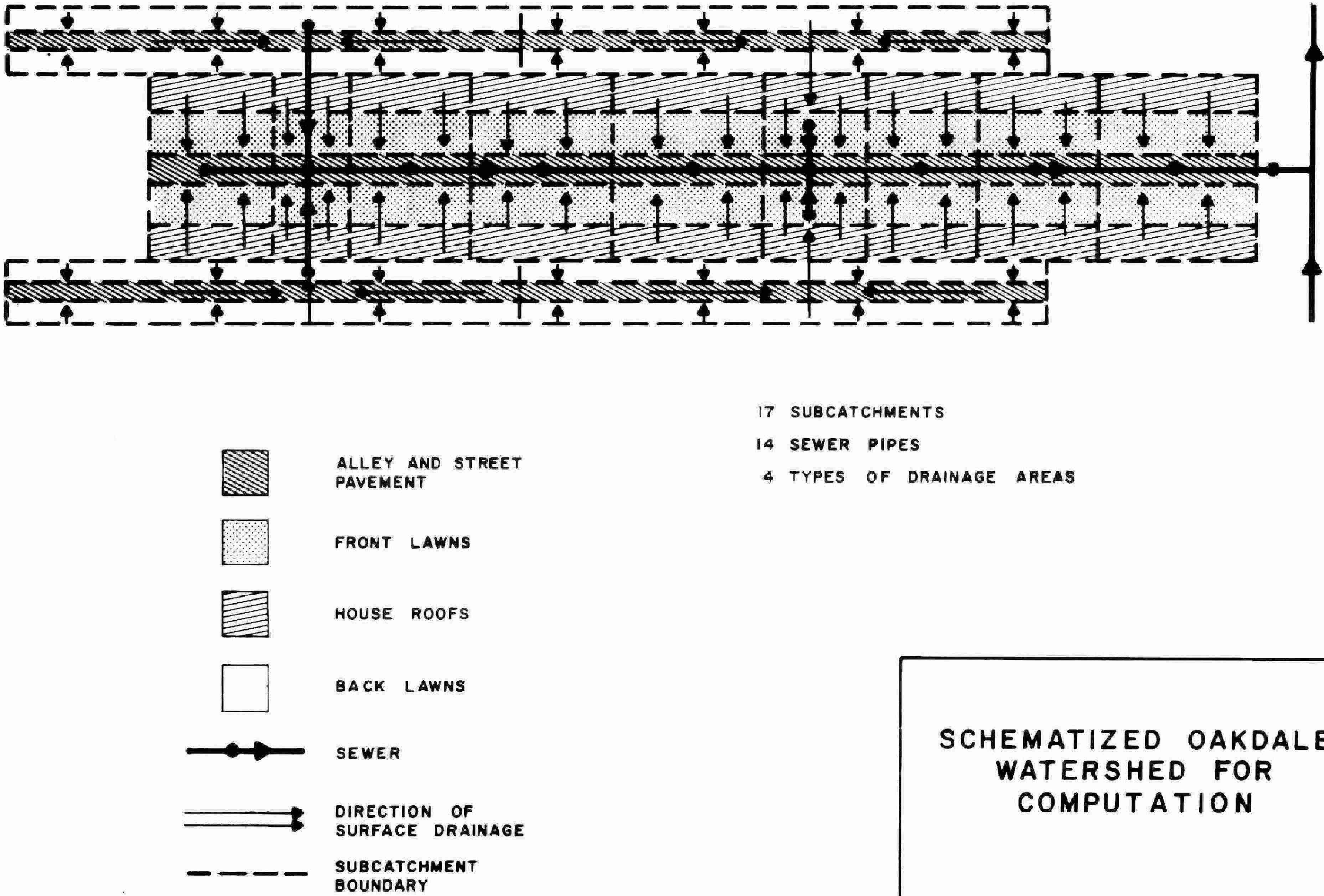


AREA = 12.95 acres

45.8% impervious

OAKDALE TEST AREA
CHICAGO, ILLINOIS
(77)

Figure 7



SCHEMATIZED OAKDALE WATERSHED FOR COMPUTATION

Fig. 8

rainfall hyetograph, I

rainfall reduction for surface storage

$$i = I \times p$$

estimate total travel time

$$T = t_i + \frac{L}{V_{full}}$$

compute time vs contributing area curve

$$a_i = \frac{A}{T} \times \Delta t$$

compute runoff hydrograph

$$Q_1 = a_1 i_1$$

$$Q_2 = a_2 i_1 + a_1 i_2$$

$$Q_3 = a_3 i_1 + a_2 i_2 + a_1 i_3$$

etc.

add upstream hydrograph and route hydrograph through pipe segment

$$\frac{\Delta t}{2} (Q_{1\ in} + Q_{2\ in} - Q_{1\ out}) + S_1$$

$$= \frac{\Delta t}{2} Q_{2\ out} + S_2$$

All pipes considered?

No.

Yes

Outlet Hydrograph

i = effective rainfall
I = initial rainfall
p = % reduction factor

T = total travel time
t_i = inlet time

L = length of pipe segment
V_{full} = full-flow velocity

a_i = incremental area
A = total impervious area tributary to pipe segment
Δt = time increment
Q = runoff hydrograph

Q_{in} = inflow hydrograph
Q_{out} = outflow hydrograph
S = pipe storage from constant depth assumption or discharge - storage relationship

ALGORITHM OF THE RRL MODEL

Fig. 9

rainfall hyetograph r(t)

deduct infiltration

$$i_1(t) = r(t) - f(t)$$

$$f(t) = f_c + (f_o - f_c)e^{-at}$$

deduct depression storage

$$i(t) = i_1(t) - d(t)$$

linear reservoir pulse response for one minute intervals defined as

$$u(1, t) = \int_{t-1}^{t} \frac{1}{t_o} e^{\tau/t_o} d\tau$$

inlet hydrograph ordinates are derived

$$q(t) = \int_o^t i(t-\tau)\, u(1,\tau)\, d\tau$$

total inlet hydrograph

$$Q(t) = \Sigma(q_i(t) A_i)$$

all area types considered? — No (return to total inlet hydrograph)

average discharge over central 50% of summed hydrograph computed (representative discharge) and velocity calculated

$$V_R = \frac{1.49}{n} R_R^{2/3} S^{1/2}$$

hydrograph is routed by time offset method

$$T_{OFF} = \frac{L}{V}$$

all pipes considered? — No (return to average discharge); Yes

OUTLET HYDROGRAPH

r(t) = rainfall hyetograph

i(t) = effective rain

f(t) = infiltration capacity

f_c = final infiltration rate

f_o = initial infiltration rate

a = decay coefficient

d(t) = depression storage filling rate

t_o = constant

t = time increment

q(t) = inlet hydrograph/unit area

Q(t) = inlet hydrograph

A_i = contributing area of type i

V_R = representative velocity

R = hydraulic radius corresponding to representative discharge

S = slope of conduit

n = Manning's roughness coefficient

AGORITHM OF THE QUURM MODEL

Fig. 10

routing procedure is more complex, the same method being used in the basic subcatchment and the main sewer.

Another proprietary model described in publications which were not available until after the commencement of this study is that developed at the Massachusetts Institute of Technology (MIT). This model has a more sophisticated overland flow subroutine than the other models considered (except perhaps the Dorsch model). A limited comparison with the SWMM was possible on the basis of published data.

For each of the SWMM, RRL, UCUR and Unit Pulse models, computer program changes and improvements were incorporated during the course of the study. Different shematization and input methods were also studied.

3.2.2 Previous comparisons of simulated and measured runoffs for the studied models

Authors of all models considered in the study have included a comparison between their model and measurements in their original publications (14, 56, 82). Comparisons considering only a few storms on a couple of test areas are useful for the calibration of parameters but are not very conclusive as a test of a model's performance as indicated in Subsection 3.1.2. At the same time, none of the available papers on hydrograph models indicate whether or not the measurements were divided into two groups, one for calibration and one for validation.

At the outset of this study, the available literature on the SWMM indicated very good agreement between computed and measured runoff hydrographs for two storms on one test area and one storm on another area (14). Subsequent comparisons for only one storm on a different very large area were less favourable (69). At the same time, the authors of the UCUR model presented a comparison for only two storms on a large test area with many subcatchements. Again the agreement was excellent (56). Available publications about the Dorsch HVM model also indicate comparison for two storms on two test areas (41). The available publication about the MIT model indicates comparisons for urban watersheds at only one test area using three storms (44).

The RRL model is the only one for which considerable test results were available. Both in the original publication by Watkins (82) and a later study by Terstriep and Stall (73), results for a large number of storms and many test areas are presented along with a comprehensive discussion of the model itself. The first one found an acceptable agreement for 14 watersheds in England. The excess rain used in all these comparisons was determined by measurements which indicates implicitly an effort for calibration. The second study, which did not consider a similar rainfall analysis, found a much poorer agreement on 10 test areas in the U.S.A. Measured flows were in general larger than those computed. This difference was explained by the much smaller percentage of imperviousness and the higher rainfall intensity characteristics of North American conditions. The same areas were not used, however, for comparisons with other hydrograph models.

After this study began, a paper was published by Papadakis and Preul (54) comparing the RRL and SWMM models and their own UCUR model on the Oakdale Chicago test area, again for only two storms. All of the models are shown to reproduce the recorded hydrograph quite well but the UCUR results are somewhat better in both cases. The authors state that the improved hydrograph from the UCUR model probably resulted from reduced infiltration and surface detention parameters used with this model to account for wet antecedent conditions. The results of the SWMM and RRL models were obtained from the previously mentioned publications and not derived by the authors.

In the same paper, the SWMM and UCUR models are compared for two storms on the 2,380 acre Bloody Run test area of Cincinnati. This time the UCUR results were very good while the SWMM results (again from a previous study [69]) were poor. The data for the UCUR model, however, include infiltration parameters based on the Antecedent Precipitation Index and field measurements. The SWMM results were derived using default parameters for infiltration which differ from the UCUR parameters. Accuracy of the data collected for this area is also questionable because all 3 of the rain gauges were located beyond the boundaries of

this large catchment. The system schematizations for the UCUR and SWMM models are different due to the requirements of the computer programs, making comparisons of this nature difficult. Also, the subcatchments of up to 250 acres used in this test area may be considered too large for such an assessment. The contradictory results discussed above justify the need for an independent evaluation of these hydrograph models. While this study was in progress, it was learned that similar activities are underway in the U.S.A. and Australia.

An independent evaluation of the RRL, UCUR and SWMM models is being done by D.P. Heeps et al at Monash University in Australia (30) for a 172 acre test area in Melbourne. Preliminary results using the three models for one 20 hour storm indicate that the SWMM model gives the best results with the UCUR models giving consistently high flows and the RRL model lower flows. It was found, however, that the UCUR model required only about one-half of the computer processing time needed by the SWMM model for this area. This was possible by a modification of the original program. It is understood that the study in Australia is continuing.

Another study being carried on at present is that of Battelle Northwest Research Institute. Battelle analyzed 17 models from a theoretical viewpoint and retained seven for a more detailed analysis including the Dorsch HVM, UCUR, SWMM, MIT models, a model developed by the Chicago Metropolitan District and one developed by Battelle. Attempts were being made to include also the Hydrocomp and La Salle Models but by the end of March 1974 the only proprietary model data made available to Battelle were those of Dorsch used also in this study. Battelle also found inconsistencies in measurements from large test areas, including Bloody Run in Cincinatti which was used in previous assessments (69). Conclusions will be available only at the end of this research.

It is felt that, considering the present state-of-the-art, the scope of any assessment of this kind should not include the strong recommendation of one particular model. The most widely used model at present is the Rational Method and the primary problem is to determine whether or not the runoff hydrograph methods offer definite advantages

compared with that method and to make operational computer programs available to Canadian practice. Advantages of a given model for various applications may be pointed out, but it is felt that advocation of a generally "best" model is not appropriate.

3.2.3 Methodology of assessment of models used for this study

Previously, model evaluations have been done for a small number of events and on a limited number of subcatchments. The few events considered were sometimes those used for the calibration of one of the models which evidently would bias any comparisons with other models. Parameters and schematization used for the various models also differed. It was attempted, therefore, to base this assessment on the simulation of many of the available events and, as far as possible, by an homogenous selection of the model parameters. This could be done in two ways:

a) Optimizing the parameters for each model and comparing the results after optimization.

b) Selecting common parameters for all models on the basis of recommended values of the model builders.

The first method improves each model as far as possible according to the available data. It was considered, however, that the models should be compared from the viewpoint of the municipal engineer who would apply the model without having a large set of data for calibration. It was therefore considered that the second method, which is in fact a validation of recommended procedures for additional storms on new areas, would be applied. This was done for the SWMM, RRL and UCUR models. An optimization procedure was applied, however, for the SWMM model to determine the potential benefits of improvement of the parameters and comparison with the QUURM model where additional calibration has been done by the author.

Several methods were considered to assess the relative performance of each model:

(i) A method suggested by the University of Purdue which takes into consideration all the points of the hydrograph including the shape and volume as described in Appendix

III and Table 3 (64).

(ii) A comparison of peak flow and time to peak only.

While the first method of comparison is more exacting, since it takes into consideration every point on the hydrograph, in many cases from a practical viewpoint only the peak characteristics of the hydrograph and the total volume of runoff are of importance. It was decided, therefore, to apply both methods in this study. The assessment was also based on the theoretical analysis of the different subroutines, which tried to explain the comparative performance. Also assessed were other factors such as:

1) Effort required in input description, data collection and flexibility of schematization.
2) Computer cost.
3) Availability, current status and potential for improvement.

3.3 Comparison of Rainfall-Runoff Simulation by the RRL, SWMM and and UCUR Models

3.3.1 Comparative study for the Oakdale test area (Chicago)

The first tests were done using measurements in the Oakdale residential district of Chicago (77).

The Oakdale Avenue test area is a 12.95 acre residential district in Chicago, Illinois, consisting of a homogeneous single family type development and a rectangular grid street plan. The combined sewer outlet is 30 inches in diameter and all of the houses are directly connected to the sewer. The area is 45.8% impervious. A detailed plan of this catchment is shown in Figure 7.

Runoff measurements were carried out between 1959 and 1968 using a 30-inch parabolic flume located in a measuring vault at the downstream end of the sewer where it is connected to a large trunk sewer. A tipping bucket rain gauge was used to measure rainfall and both this rain gauge and a transmitter from the flow measuring vault were connected to remote recorders over leased telephone lines. All instrumentation operated only during periods of rainfall.

Thirteen storms were selected for the purposes of this study. Events for which the runoff measurements were incomplete or the maximum flows were less than 4 cfs were not considered.

TABLE 3. RATINGS OF THE STATISTICAL MEASURES OF THE AGREEMENT COMPUTED AND RECORDED HYDROGRAPHS [64]

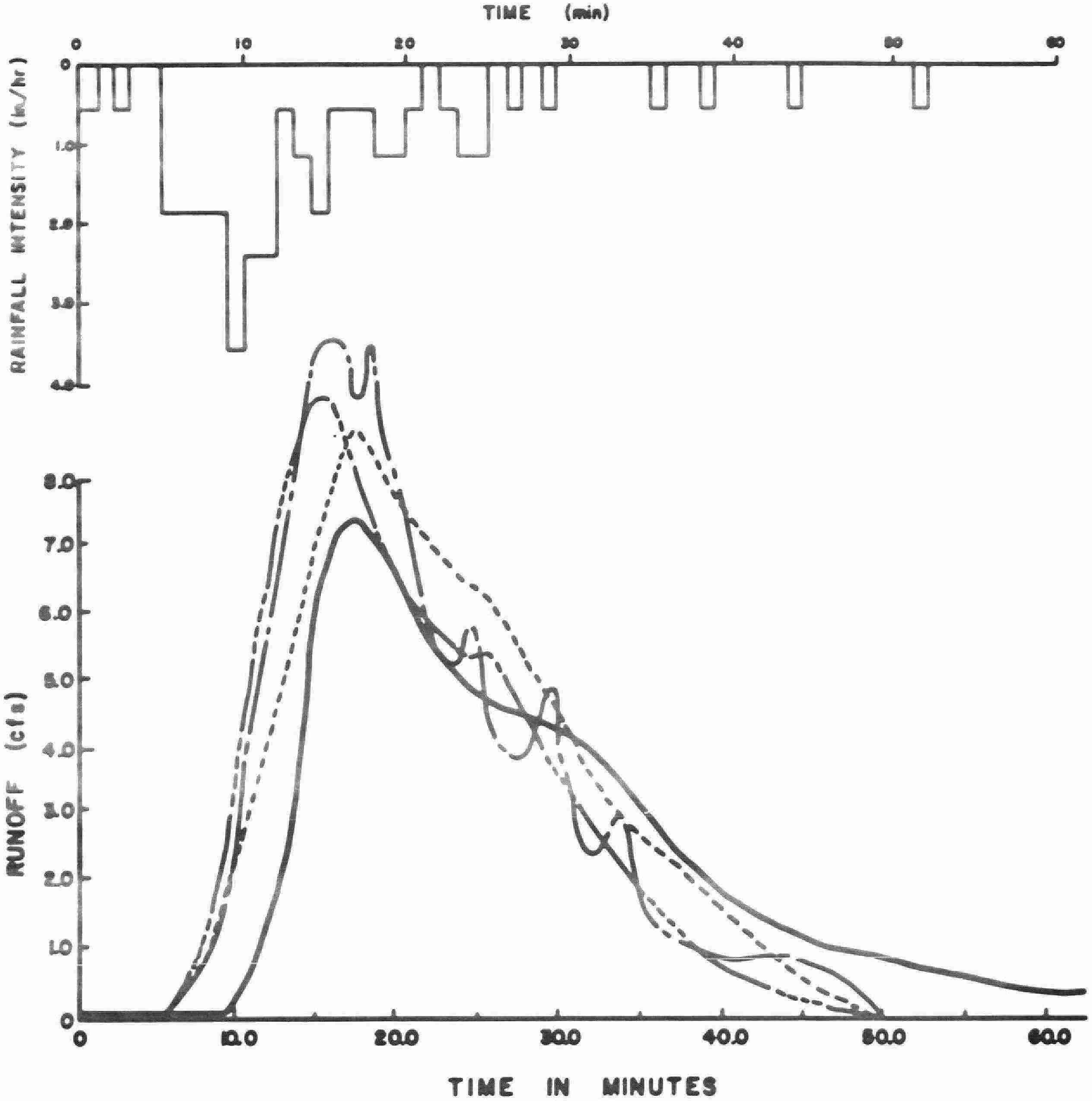
Correlation Coefficient (R)	Rating
$0.99 \leq R < 1.0$	Excellent
$0.95 \leq R < 0.99$	Very Good
$0.90 \leq R < 0.95$	Good
$0.85 \leq R < 0.90$	Fair
$0.00 \leq R < 0.85$	Poor
Special Correlation Coefficient (R_s)	
$0.99 \leq R_s < 1.0$	Excellent
$0.95 \leq R_s < 0.99$	Very Good
$0.90 \leq R_s < 0.95$	Good
$0.85 \leq R_s < 0.90$	Fair
$0.00 \leq R_s < 0.85$	Poor
Integral Square Error (ISE)	
$0\% < ISE \leq 3.0\%$	Excellent
$3.0\% < ISE \leq 6.0\%$	Very Good
$6.0\% < ISE \leq 10.0\%$	Good
$10.0\% < ISE \leq 25.0\%$	Fair
$25.0\% < ISE$	Poor

The selected rainfall events for Oakdale are characterized by long durations (up to 350 min) and low intensities. The maximum average intensity for a duration of 14 minutes, equal to the time to concentration, varied from 0.7 in/hr to 2.8 in/hr. The low intensity of these storms is probably the result of surcharge conditions occurring in the downstream collector system for larger storms, thereby causing flooding of the measurement vault and flume.

From the thirteen storms selected two have been used by previous authors (14, 56) who obtained, in general, an excellent agreement between the calculated and measured hydrographs. The first tests were conducted for these storms in order to compare the results of this study with those published by the model builders. This early comparison done during the study led to improvements in the programs and input descriptions, and finally results comparable to those published were obtained.

The SWMM, RRL and UCUR model programs were then run for the thirteen storms. The agreement between measured and computed values was for some of these storms only fair. The good agreement for a limited number of storms might be explained by the use of these storms in the selection of parameters recommended by the model builders. The additional comparisons made during this study validate these parameters but show at the same time, however, the real approximation of the methods. Comparisons for two selected storms which are typical of the overall results are given in Figures 11 and 12. Comparative results for all the peak flows and the times to peak are given in Tables 4 and 5.

Figures 13 and 14 compare the calculated and observed peak flows and times to peak respectively. Lines indicating an error of $\pm$ 20% were superimposed on the plots. One may conclude that all three models reproduce the time to peak very well for this test area. The scatter, however, for the peak flows is larger. (See Section 3.4.1) Mean values for the ratios of measured and computed peak flows and times to peak are given below. The results of the point by point comparison of the measured and computed hydrographs is shown in Table 6.



COMPARISON of HYDROGRAPHS

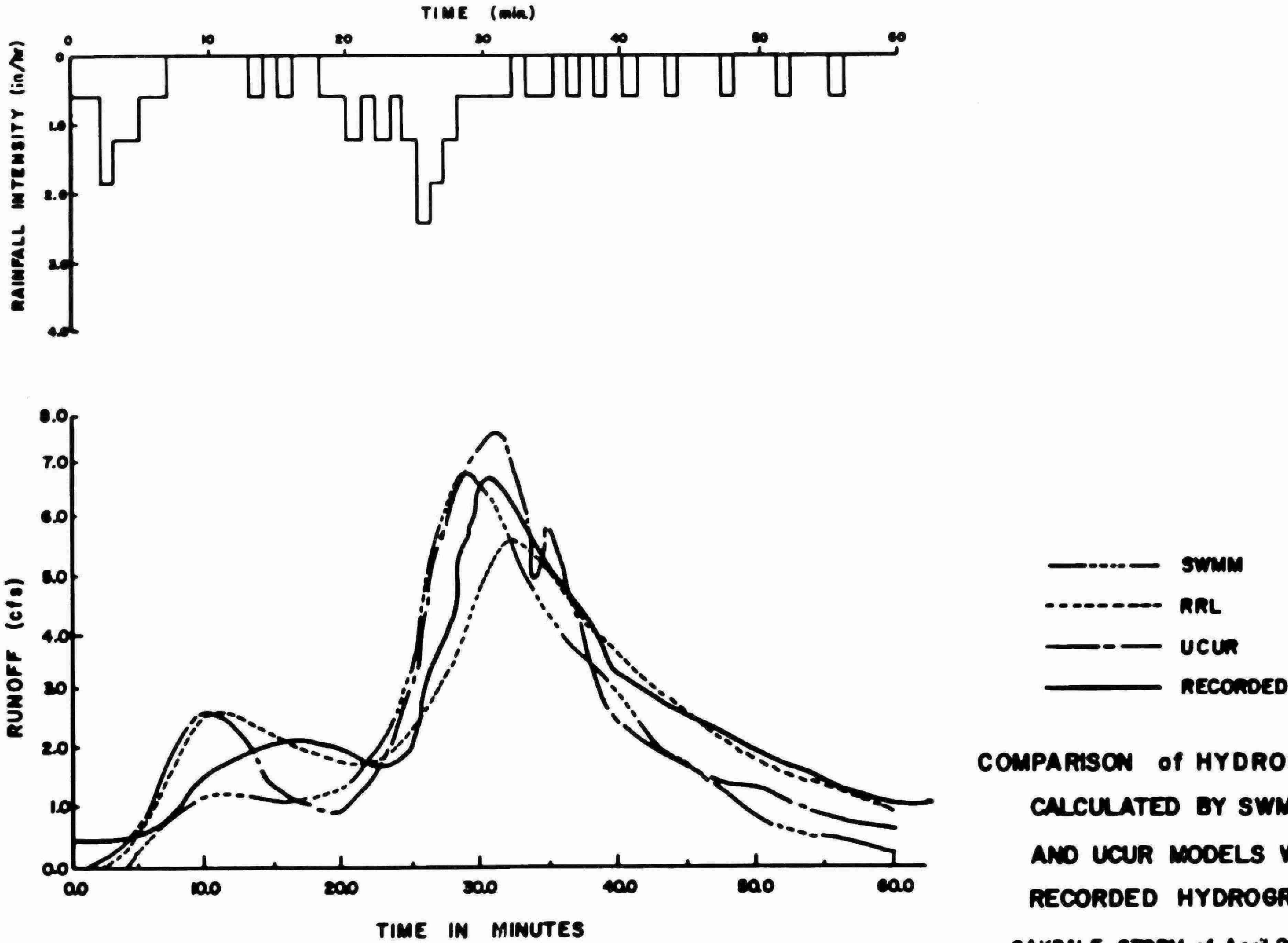
CALCULATED BY SWMM, RRL

AND UCUR MODELS WITH

RECORDED HYDROGRAPH

OAKDALE STORM. of May 19, 1959

FIG. 11



COMPARISON of HYDROGRAPHS
CALCULATED BY SWMM, RRL
AND UCUR MODELS WITH
RECORDED HYDROGRAPH
OAKDALE STORM of April 29, 1963

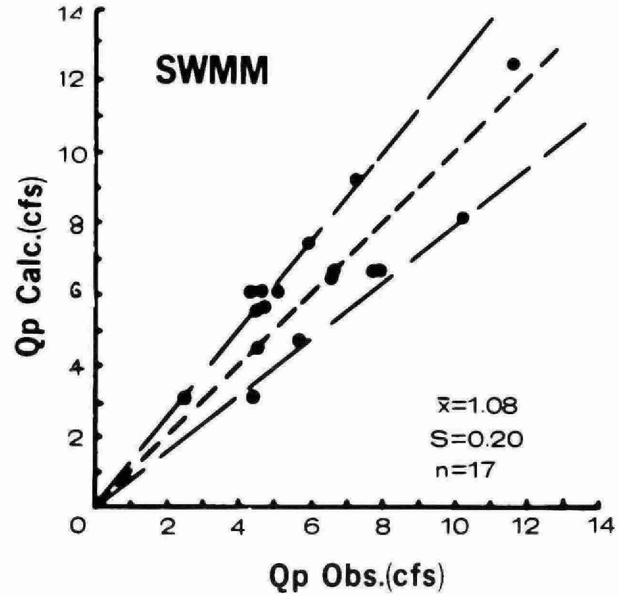
FIG. 12

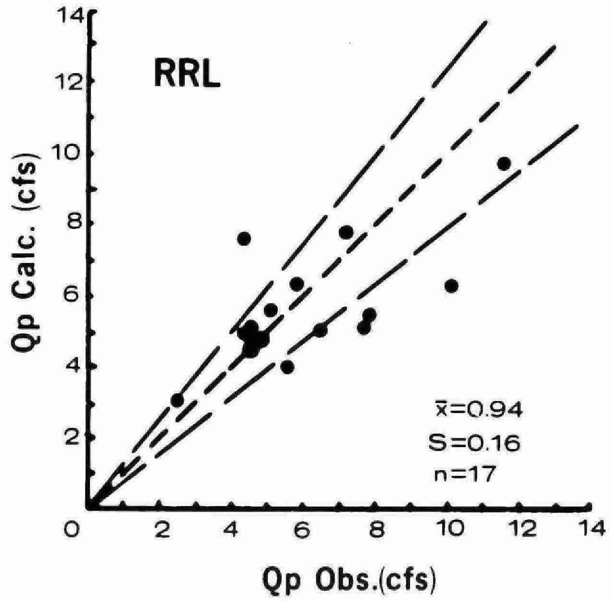
TABLE 4. RECORDED PEAK FLOWS FOR OAKDALE AND COMPUTED PEAK FLOWS OF THE SWMM, UCUR AND RRL MODELS

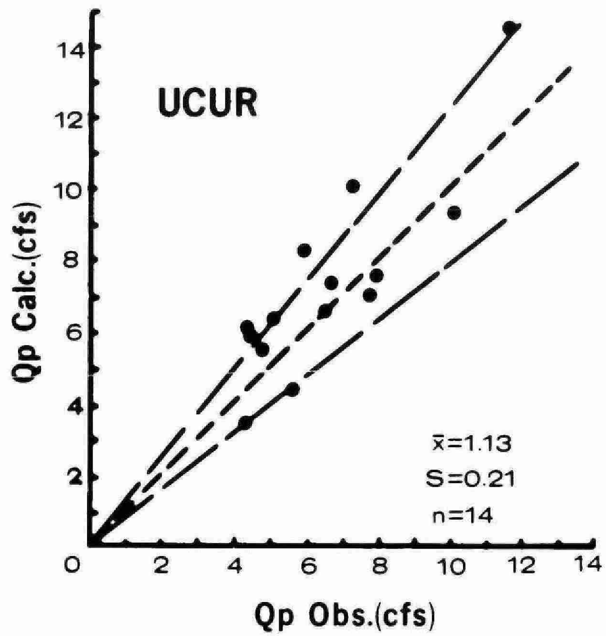
No.	STORM DATE	MEAS. Qp 1 (cfs)	Qp 2 (cfs)	2/1	UCUR Qp 3 (cfs)	3/1	RRL Qp 4 (cfs)	4/1
1	5/19/59	7.25	9.15	1.26	10.14	1.4	8.60	1.18
2	7/2/60	4.60	4.35	0.95	-	-	5.24	1.14
3	7/26/60-1	2.50	2.22	0.89	-	-	3.43	1.37
	-2	4.30	3.17	0.74	3.01	0.70	2.77	0.65
4	9/18/60	5.10	6.07	1.19	5.65	1.11	5.96	1.17
5	10/14/60	4.50	5.05	1.12	6.10	1.36	5.75	1.28
6	7/2/62-1	10.10	8.22	0.81	6.87	.68	6.73	0.67
	-2	7.90	6.74	0.85	7.24	.92	5.76	0.73
7	4/17/63-1	4.60	6.07	1.32	-	-	5.30	1.15
	-2	6.50	6.47	1.00	6.88	1.06	6.41	0.97
8	4/19/63	11.60	14.51	1.25	13.65	1.18	11.09	0.96
9	4/29/63	6.70	6.88	1.03	7.47	1.11	5.41	0.81
10	4/30/63	5.75	4.81	0.84	4.89	.85	4.22	0.73
11	6/19/63	7.80	6.08	0.78	6.46	.83	6.10	0.78
12	8/2/63-1	4.85	5.66	1.17	3.70	.76	5.24	1.08
	-2	5.95	7.62	1.28	6.20	1.04	6.80	1.14
13	9/22/64	4.45	6.19	1.39	5.83	1.31	5.03	1.13
$\bar{X}$ = MEAN $\frac{\text{Qp calc}}{\text{Qp meas}}$				1.05		1.02		1.00
S = STANDARD DEVIATION of Qp ratios				0.20		0.23		0.22

TABLE 5. RECORDED TIMES TO PEAK FOR OAKDALE AND COMPUTED TIMES TO PEAK OF THE SWMM, UCUR AND RRL MODELS.

No.	STORM DATE	MEAS. Tp 1 (mins)	SWMM Tp 2 (mins)	SWMM $^{2}/_{1}$	UCUR Tp 3 (mins)	UCUR $^{3}/_{1}$	RRL Tp 4 (mins)	RRL $^{4}/_{1}$
1	5/19/59	17	15	0.88	15	.88	17	1.00
2	7/2/60	14	14	1.00	-	-	14	1.00
3	7/26/60-1	11	7	0.64	-	-	10	0.91
	-2	210	209	1.00	207	.99	211	1.00
4	9/18/60	27	22	0.81	24	.89	24	0.89
5	10/14/60	16	10	0.63	11	.69	14	0.88
6	7/2/62-1	92	92	1.00	95	1.03	95	1.03
	-2	162	162	1.00	164	1.01	165	1.02
7	4/17/63-1	20	14	0.70	-	-	16	0.80
	-2	103	101	0.98	102	.99	103	1.00
8	4/19/63	42	40	0.95	38	.90	44	1.05
9	4/29/63	30	29	0.97	30	1.00	33	1.10
10	4/30/63	288	291	1.04	291	1.01	293	1.02
11	6/19/63	11	8	0.73	7	.64	12	1.08
12	8/2/63-1	18	16	0.89	16	.89	18	1.00
	-2	90	89	0.97	90	1.00	91	1.01
13	9/22/64	38	36	0.95	35	.92	38	1.00
$\bar{X}$ = MEAN $\frac{\text{Tp calc}}{\text{Tp meas}}$				0.92		.92		.99
S = STANDARD DEVIATION of Tp ratios				0.14		.18		0.07







Qp = peak flow

x̄=mean of $\frac{\text{Qp(calc)}}{\text{Qp(obs)}}$

n=no. of observations

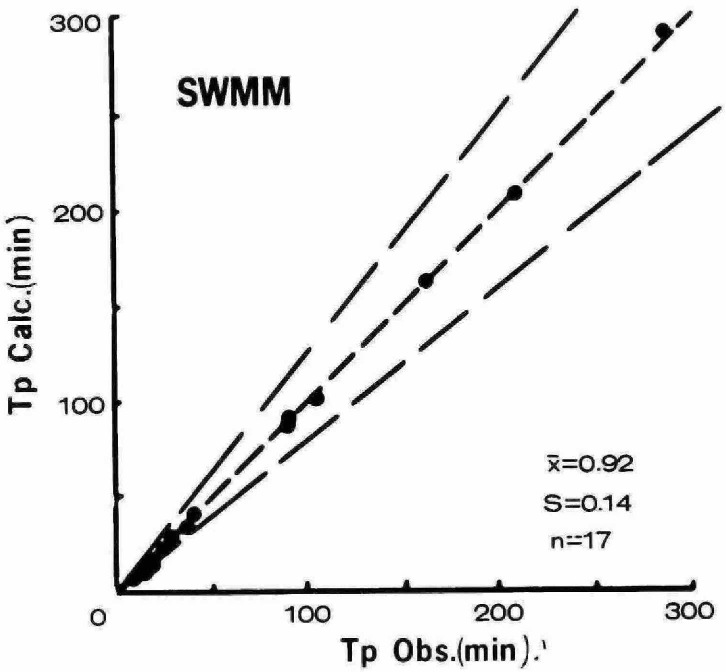
Qp(obs)=Qp(calc) – – – – –

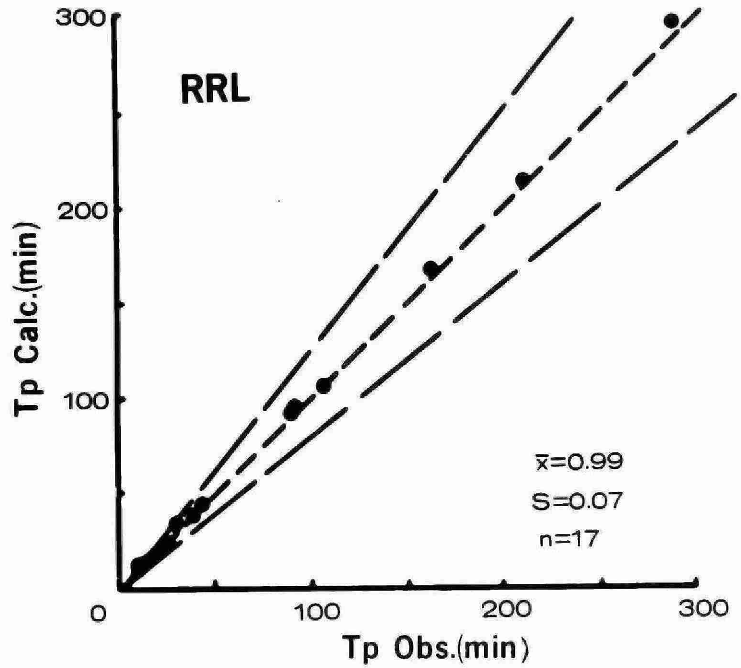
Lines of ± 20% error —— ——

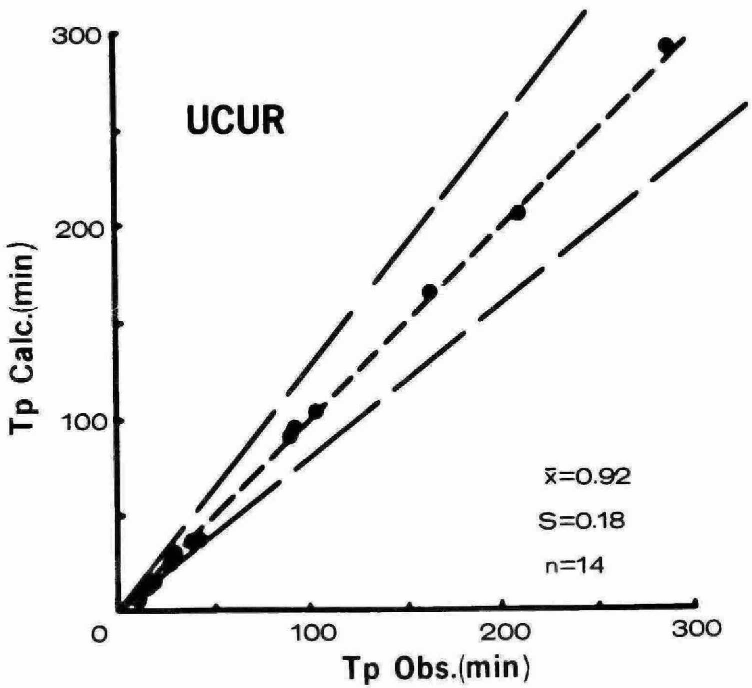
OAKDALE, CHICAGO

Qp CALCULATED vs. Qp OBSERVED SWMM, RRL AND UCUR MODELS

Figure 13







Tp=time to peak

$\bar{x}$=mean of $\frac{Tp(calc)}{Tp(obs)}$

n=no. of observations

Tp(obs)=Tp(calc) — — — — —

Lines of $\pm$ 20% error —— ——

OAKDALE, CHICAGO

Tp CALCULATED vs. Tp OBSERVED

SWMM, RRL AND UCUR MODELS

Figure 14

TABLE 6. STATISTICAL COMPARISON OF RECORDED RUNOFF HYDROGRAPHS ON THE OAKDALE TEST AREA WITH THE SWMM, RRL AND UCUR COMPUTED HYDROGRAPHS

No.	STORM DATE	PARA-METER	SWMM	RRL	UCUR
1	5/19/59	R	0.988	0.981	0.935
		RS	0.978	0.961	0.801
		ISE	1.76	2.36	7.72
2	7/2/60	R	0.982	0.942	0.932
		RS	0.949	0.841	0.902
		ISE	2.95	5.21	5.34
3	7/26/60-1	R	0.957*	0.903*	-
		RS	0.887	0.828	-
		ISE	2.13	2.63	-
	-2	R	-	-	.715
		RS	-	-	.677
		ISE	-	-	5.12
4	9/18/60	R	0.873	0.851	0.739
		RS	0.857	0.804	0.645
		ISE	2.94	3.44	5.60
5	10/14/60	R	0.800	0.796	0.576
		RS	0.661	0.647	0.288
		ISE	6.12	6.25	14.26
6	7/2/62	R	0.959	0.918	0.910
		RS	0.911	0.818	0.914
		ISE	2.12	3.04	2.09
7	4/17/63	R	0.873	0.892	0.856
		RS	0.811	0.810	0.748
		ISE	3.40	3.41	8.62
8	4/19/63	R	0.963	0.949	0.901
		RS	0.923	0.907	0.782
		ISE	4.38	4.84	8.36
9	4/29,30/63-1	R	0.973*	0.940*	.962
		RS	0.943	0.887	.937
		ISE	1.64	2.32	3.37
10	6/19/63	R	0.979	0.941	0.834
		RS	0.774	0.825	0.724
		ISE	8.21	7.23	9.56
11	8/2/63	R	0.976	0.979	0.909
		RS	0.953	0.956	0.813
		ISE	2.43	2.35	7.78
12	9/22/64	R	0.995	0.955	0.900
		RS	0.961	0.933	0.773
		ISE	2.77	3.63	7.17

(For explanation of parameters, see Table 3, and Appendix 3)

* Combined results for whole storm.

Results of Model Tests on Oakdale	MODEL SWMM	UCUR	RRL
Mean $\frac{\text{Qp computed}}{\text{Qp measured}}$	1.08	1.13	0.94
Standard deviation of Qp ratios	0.20	0.21	0.16
Mean $\frac{\text{Tp computed}}{\text{Tp measured}}$	0.86	0.88	1.02
Standard deviation of Tp ratios	0.18	0.38	0.10
Number of Observations	17	14	17

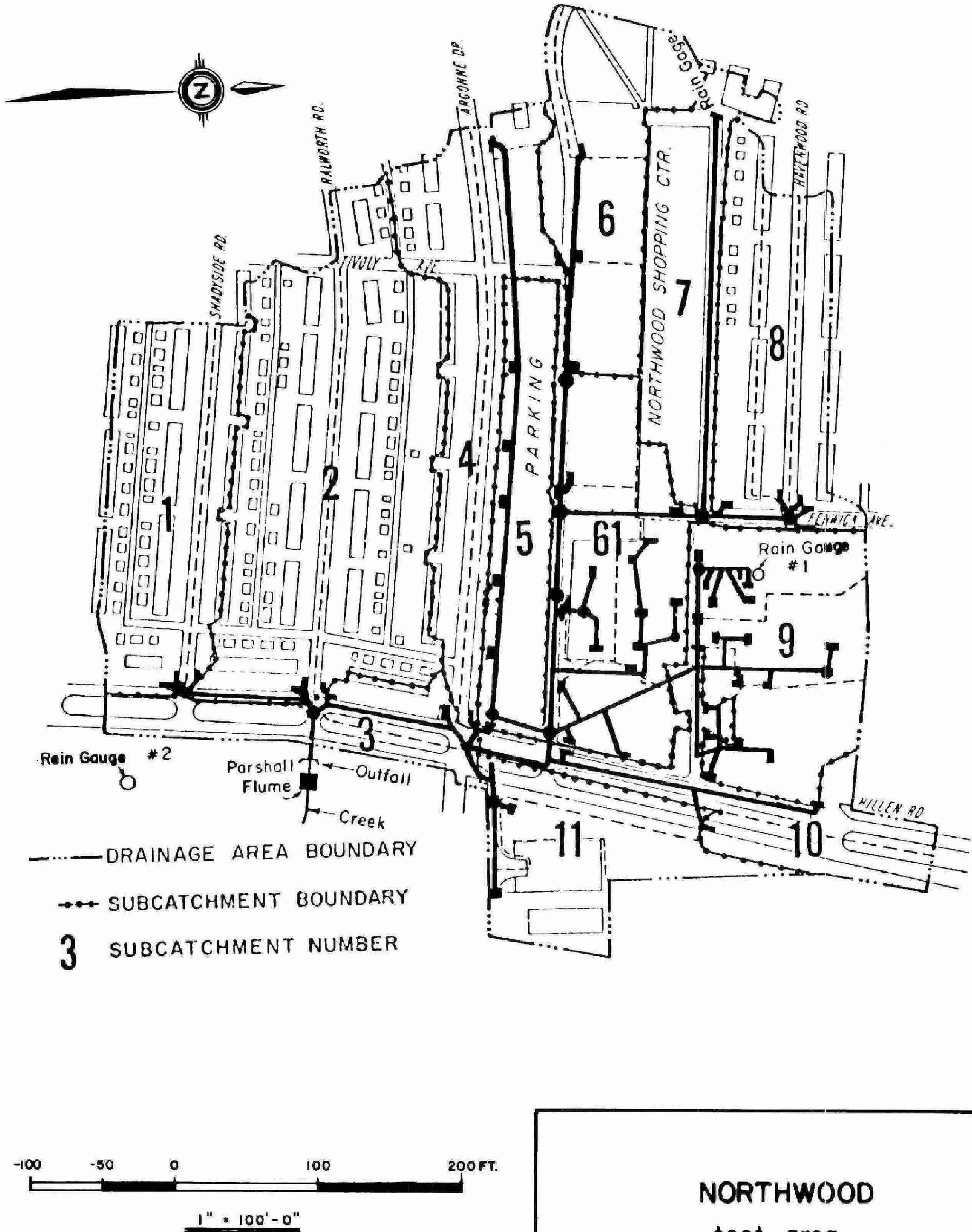
The difference in performance between the three models for this test area is considered to be comparatively small from a practical point of view, although the overall performance of the SWMM model as indicated in Table 5 appears to be somewhat better. The analysis of the output from the SWMM and UCUR models indicates that for the simulated storm conditions and assumed infiltration capacities, the contribution from the pervious areas is negligible, which means that this basic assumption of the RRL method is quite accurate in this particular case. An attempt was made to improve these results by considering antecedent conditions and rainfalls obtained through the cooperation of the City of Chicago. However, no significant correlation between antecedent conditions and relative error was found.

3.3.2 Comparative study for the Northwood test area (Baltimore)

The next test area analyzed was the Northwood Baltimore catchment which again was used by some of the model builders in their publications (76).

This is a 47.4 acre drainage area consisting of a 17.4 acre shopping centre and 30 acres of residential development. The residential buildings consist entirely of group housing, with 3 or 4 houses per group. The total imperviousness is 68% and the outfall drain is a 48 inch diameter reinforced concrete pipe. A detailed plan is shown in Figure 15.

Measurements were begun in 1959 using a weighing bucket rain gauge and a concrete Parshall flume with a 12-foot throat width down-



AREA = 47.4 acres
68 % impervious

NORTHWOOD
test area
BALTIMORE, MARYLAND
(76)

Fig. 15

stream of the outfall. In 1963, a tipping-bucket rain gauge replaced the weighing bucket rain gauge, thereby allowing closer synchronization of the rainfall and runoff measurements. The initial pulse from the rain guage activated the recording mechanism and rainfall and runoff were recorded on the same high-speed chart. The recording instruments shut off automatically after a pre-set time interval after the last rain gauge signal.

Most of the Northwood data was collected during the Summer months, with the result that the storms are generally of high intensity and short duration, with well-defined storm events. Fourteen storms were initially selected for this study.

Here again, the first tests were done for storms presented in the publications (14, 41) and the programs used gave very good results similar to those published. When all the storms were tested, however, the scatter of the results was much larger than in the Oakdale tests. (See Tables 7 and 8 and Figures 16 and 17.) The standard deviation of the ratio Qp calculated/Qp measured for the SWMM model was, for example, .34 for Northwood compared to 0.20 for Oakdale. After several unsuccessful attempts to improve the results by changing the parameters, Professor J. Shaake, who was involved in the Johns Hopkins studies, was contacted. According to information received from Dr. Shaake, the measurements from Northwood have not been analyzed by the research team and errors are possible. He also indicated that results from another test area used in the Johns Hopkins program, namely Gray Haven, may be more precise. Consequently, it was decided not to consider Northwood any further for this analysis.

3.3.3 Comparative study for the Gray Haven test area (Baltimore)

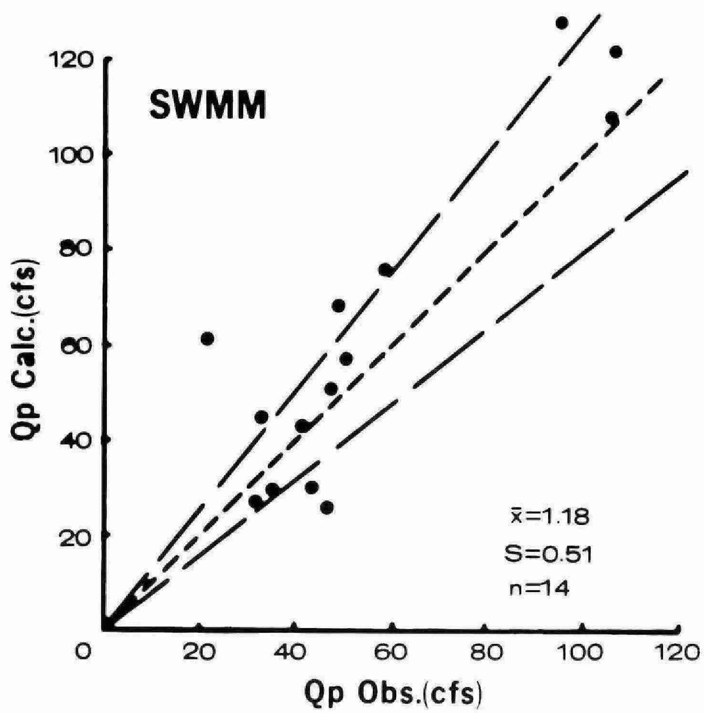
Gray Haven, Baltimore, is an area about 23.3 acres in size, of which 52% is impervious. It consists of a homogeneous residential development containing group houses. The separated storm sewer is connected to a 42 inch diameter reinforced concrete outfall. A detailed plan of the drainage catchment and a schematic layout of the drainage system is shown in Figure 18.

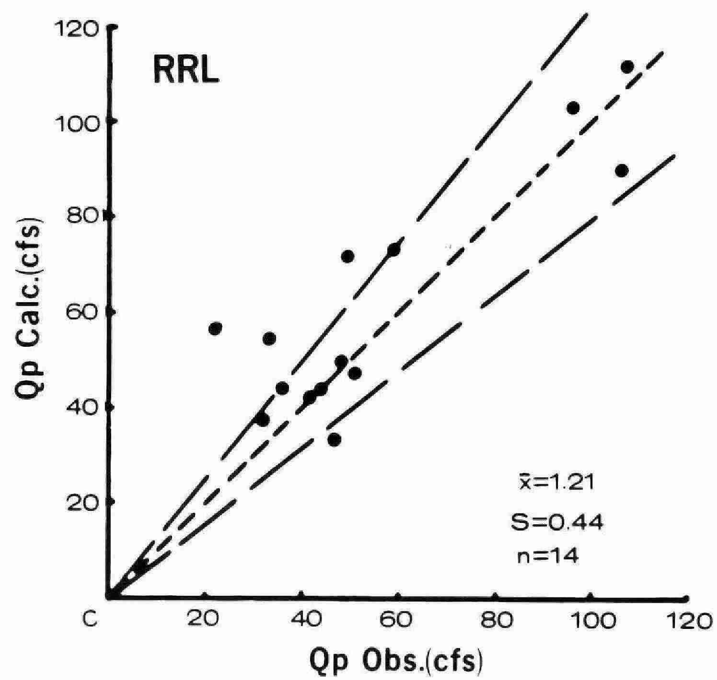
TABLE 7. RECORDED PEAK FLOWS FOR NORTHWOOD AND COMPUTED PEAK FLOWS OF THE SWMM, RRL AND HVM MODELS

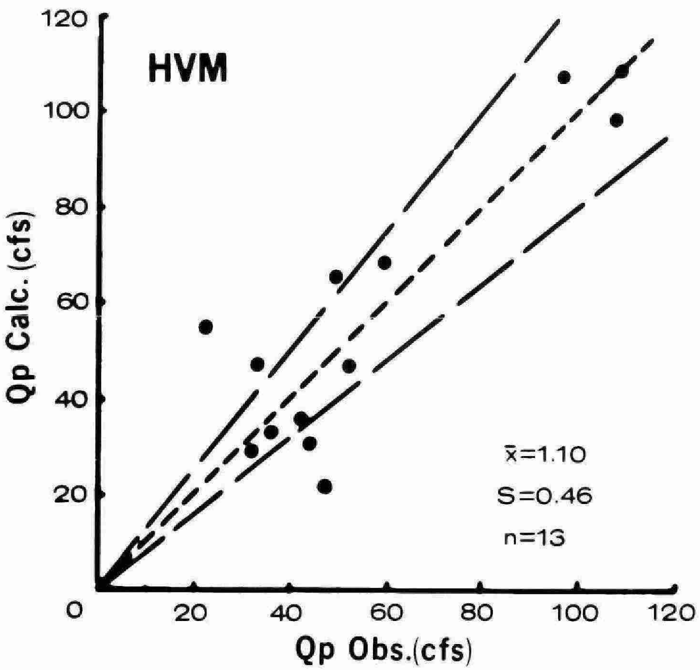
No.	STORM DATE	MEAS. Qp 1 (cfs)	SWMM Qp 2 (cfs)	SWMM 2/1	RRL Qp 3 (cfs)	RRL 3/1	HVM Qp 4 (cfs)	HVM 4/1
1	3/26/64	36.0	30.4	0.84	44.3	1.23	33.1	0.92
2	11/25/64	42.2	43.3	1.03	42.9	1.02	36.1	0.86
3	12/26/64	48.8	51.6	1.06	50.8	1.04	-	-
4	7/5/65	107.1	122.8	1.15	112.2	1.05	99.6	0.93
5	7/15/65	47.4	26.5	0.56	33.0	0.70	22.7	0.48
6	7/28/65	22.3	61.8	2.78	57.0	2.56	55.0	2.47
7	8/1/65-1	59.3	77.0	1.13	73.0	0.92	47.9	0.94
8	-2	51.2	57.8	1.30	47.1	1.23	69.6	1.18
9	8/4/65-1	96.2	128.6	1.34	103.6	1.08	108.0	1.12
10	-2	106.2	108.1	1.02	90.4	0.85	110.9	1.04
11	8/8/65	33.2	45.9	1.38	54.4	1.64	47.0	1.42
12	8/19/65	32.7	27.2	0.83	37.9	1.16	29.9	0.91
13	9/1/65	44.6	30.3	0.68	44.3	1.00	31.3	0.70
14	9/24/65	49.8	68.8	1.38	72.8	1.46	66.0	1.33
$\bar{X}$ = MEAN $\frac{Qp\ calc}{Qp\ meas}$				1.18		1.21		1.10
S = STANDARD DEVIATION of Qp ratios				0.51		0.44		0.46

TABLE 8. RECORDED TIMES TO PEAK FOR NORTHWOOD AND COMPUTED TIMES TO PEAK OF THE SWMM, RRL AND HVM MODELS.

No.	STORM DATE	MEAS. Tp 1 (mins)	SWMM Tp 2 (mins)	SWMM $^{2}/_{1}$	RRL Tp 3 (mins)	RRL $^{3}/_{1}$	HVM Tp 4 (mins)	HVM $^{4}/_{1}$
1	3/26/64	7	9	1.29	11	1.57	7	1.00
2	11/25/64	9	16	1.78	17	1.89	15	1.67
3	12/26/64	36	31	0.86	36	1.00	-	-
4	7/5/65	30	12	0.40	14	0.47	11	0.37
5	7/15/65	11	16	1.46	10	0.91	12	1.09
6	7/28/65	20	17	0.85	20	1.00	16	0.80
7	8/1/65-1	48	49	1.02	50	1.04	47	0.98
8	-2	15	15	1.00	18	1.20	14	0.93
9	8/4/65-1	32	33	1.03	35	1.09	32	1.00
10	-2	25	26	1.04	28	1.12	25	1.00
11	8/8/65	8	9	1.13	11	1.38	8	1.00
12	8/19/65	9	9	1.00	11	1.22	8	0.89
13	9/1/65	6	10	1.67	11	1.83	8	1.33
14	9/24/65	7	8	1.14	11	1.57	7	1.00
$\bar{X}$ = MEAN $\frac{\text{Tp calc}}{\text{Tp meas}}$				1.12		1.24		1.01
S = STANDARD DEVIATION of Tp ratios				0.34		0.37		0.29







Qp=peak flow

$\bar{x}$=mean of $\frac{Qp(calc)}{Qp(obs)}$

n=no. of observations

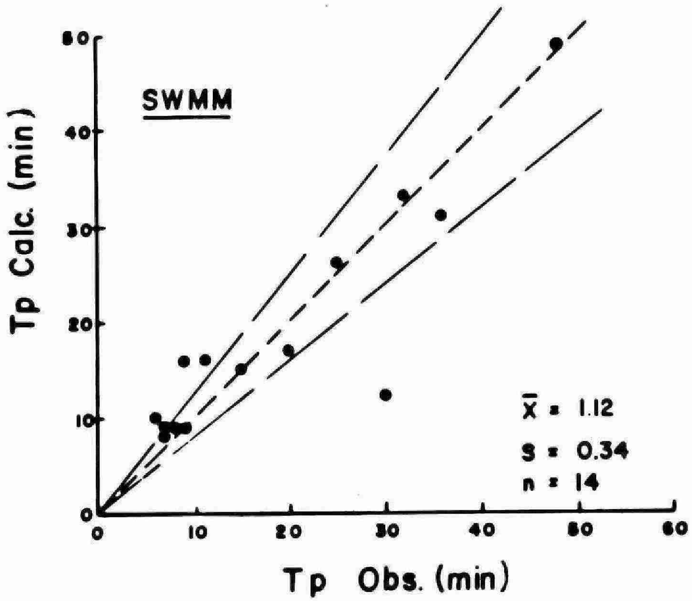
Qp(obs)=Qp(calc) – – – – –

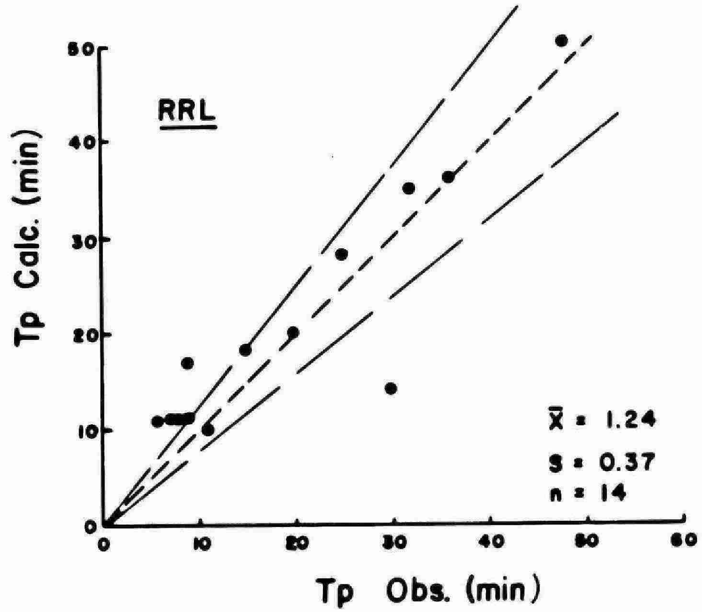
Lines of ± 20% error —— ——

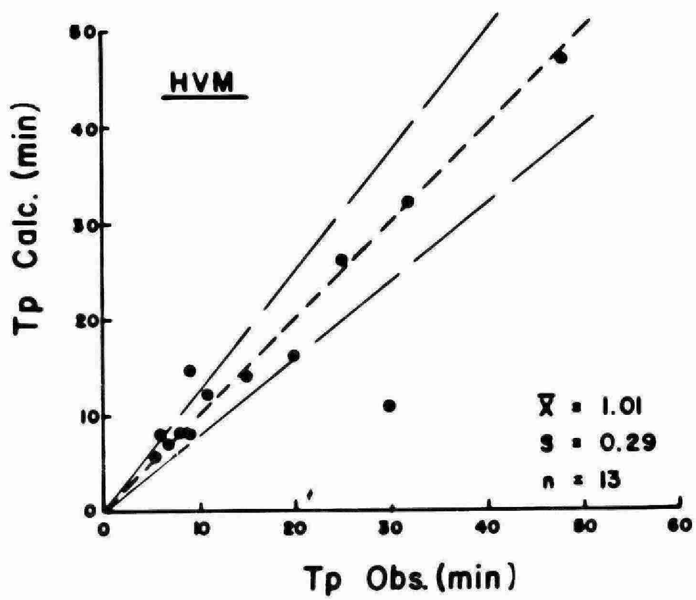
NORTHWOOD, BALTIMORE

Qp CALCULATED vs. Qp OBSERVED SWMM, RRL AND HVM MODELS

Figure 16







Tp = time to peak

$\bar{x}$ = mean of $\frac{\text{Tp (calc)}}{\text{Tp (obs)}}$

n = no. of observations

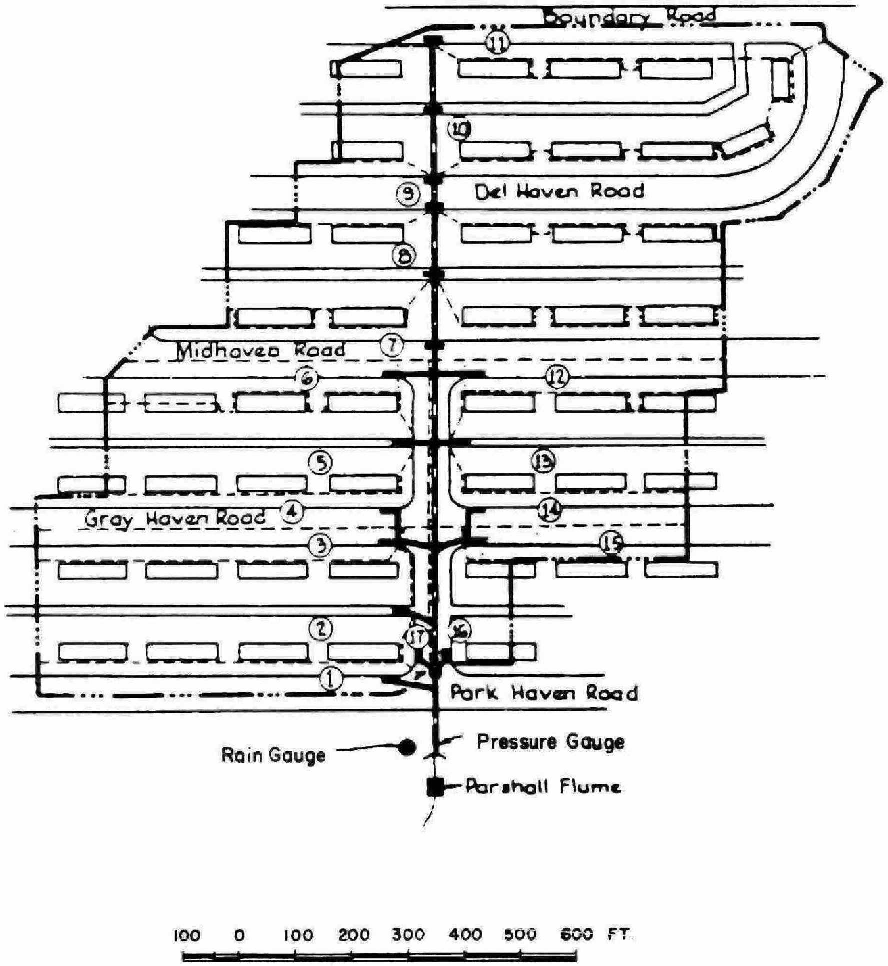
Tp (obs) = Tp (calc) — — — — —

LINES OF ±20% ERROR ——— ———

NORTHWOOD, BALTIMORE

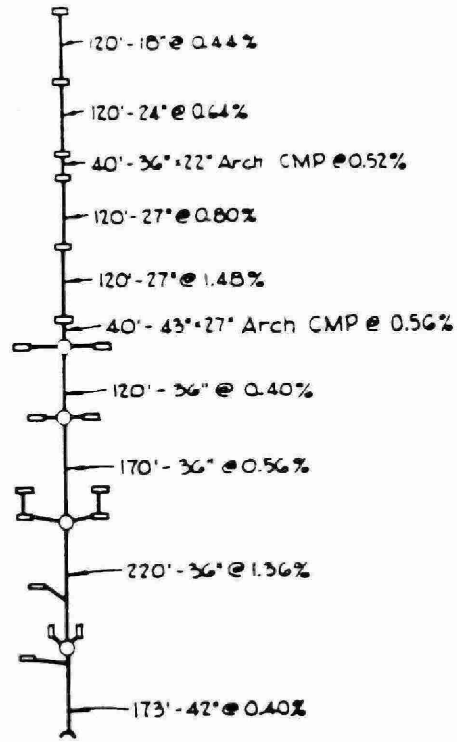
Tp CALCULATED vs. Tp OBSERVED
SWMM, RRL AND HVM MODELS

FIG. 17



AREA = 23.3 acres

52 % impervious



GRAY HAVEN TEST AREA
BALTIMORE
(44)

Fig. 18

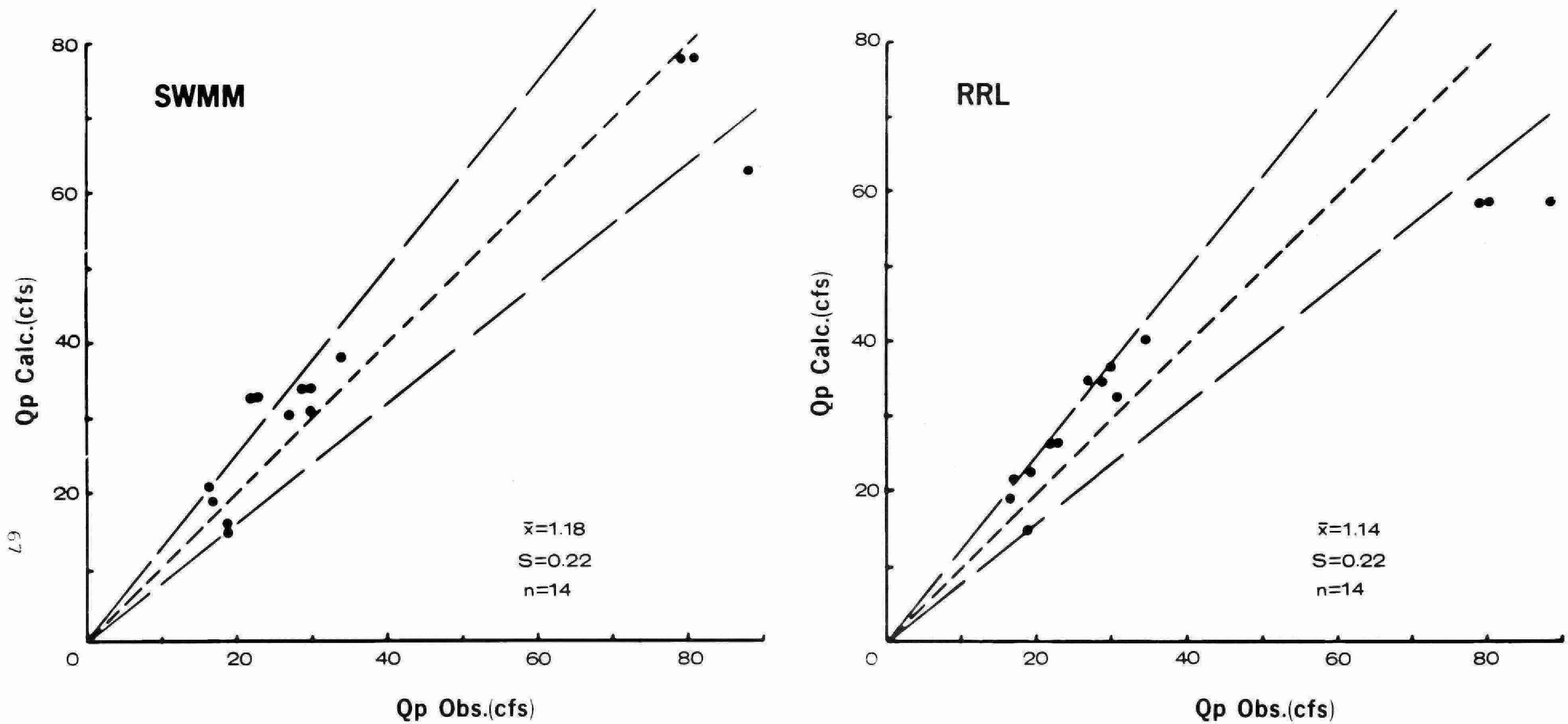
A Parshall flume and a tipping-bucket rain gauge were installed in 1962 and synchronized rainfall and runoff data were collected from 1962 to 1967. The flume has a 6-foot throat width and is made of concrete. The recording system was activated by the first pulse from the rain gauge and shut off automaticlly some time after the last rainfall signal.

Ten well-defined storm events of between 15 min and 45 min duration were chosen from the recorded data. Only major storms with a peak runoff of more than 0.8 cfs/acre were used. The maximum average storm intensities over a 10 min period, equal to the time of concentration varied from 1.4 to 5.1 in/hr.

The results of the computer runs with the SWMM and RRL models for Gray Haven compared to recorded peak flows, times to peak and runoff volumes are shown in Figures 19-21 and given in Tables 9-11. It was found again that the response for the time to peak of the SWMM is very good while that of the RRL is only fair. The errors for the peak flow are of the same order of magnitude as for Oakdale for both models. The agreement between recorded and computed volumes is also good considering that the models have not been calibrated. These results are shown below, while the results of the comparison considering the complete hydrograph are given in Table 12.

Results of Model Tests on Gray Haven	MODEL	
	SWMM	RRL
Mean $\frac{\text{Qp Computed}}{\text{Qp measured}}$	1.18	1.14
Standard deviation of Qp ratios	0.22	0.22
Mean $\frac{\text{Tp computed}}{\text{Tp measured}}$	0.98	8.89
Standard deviation of Tp ratios	0.05	0.30
Number of Observations	14	14

The difference again is small from a practical point of view. It shows, however, that while for Oakdale the RRL method gave an average Qp calculated/Qp measured smaller than 1.0, for Gray Haven it is greater than 1.0.



Qp= peak flow

$\bar{x}$=mean of $\frac{Qp(calc)}{Qp(obs)}$

n=no. of observations

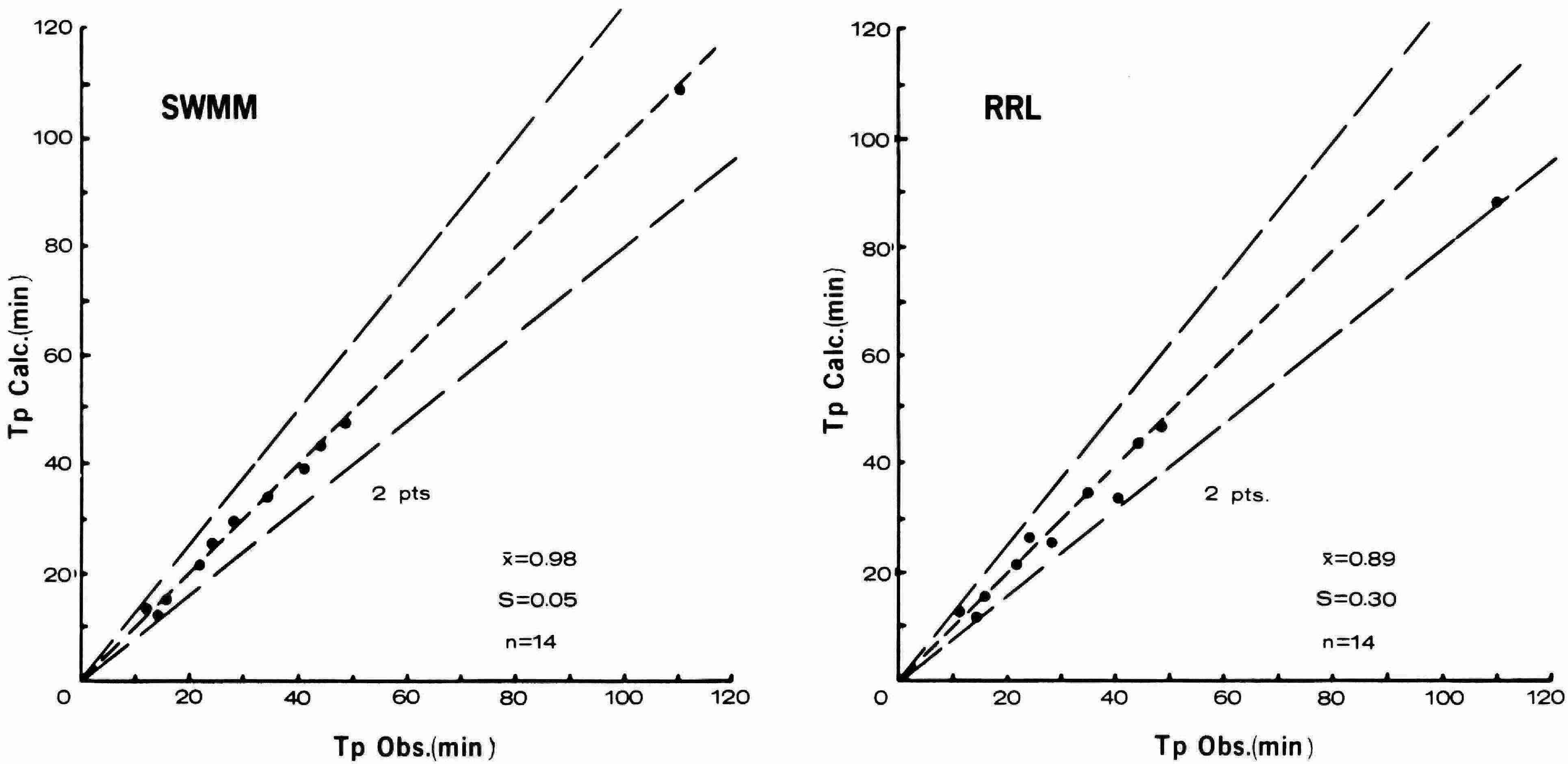
Qp(obs)=Qp(calc) – – – – –

Lines of ± 20% error —— ——

GRAY HAVEN, BALTIMORE

Qp CALCULATED vs. Qp OBSERVED SWMM AND RRL MODELS

Figure 19



Tp=time to peak

x̄=mean of $\frac{Tp(calc)}{Tp(obs)}$

n=no. of observations

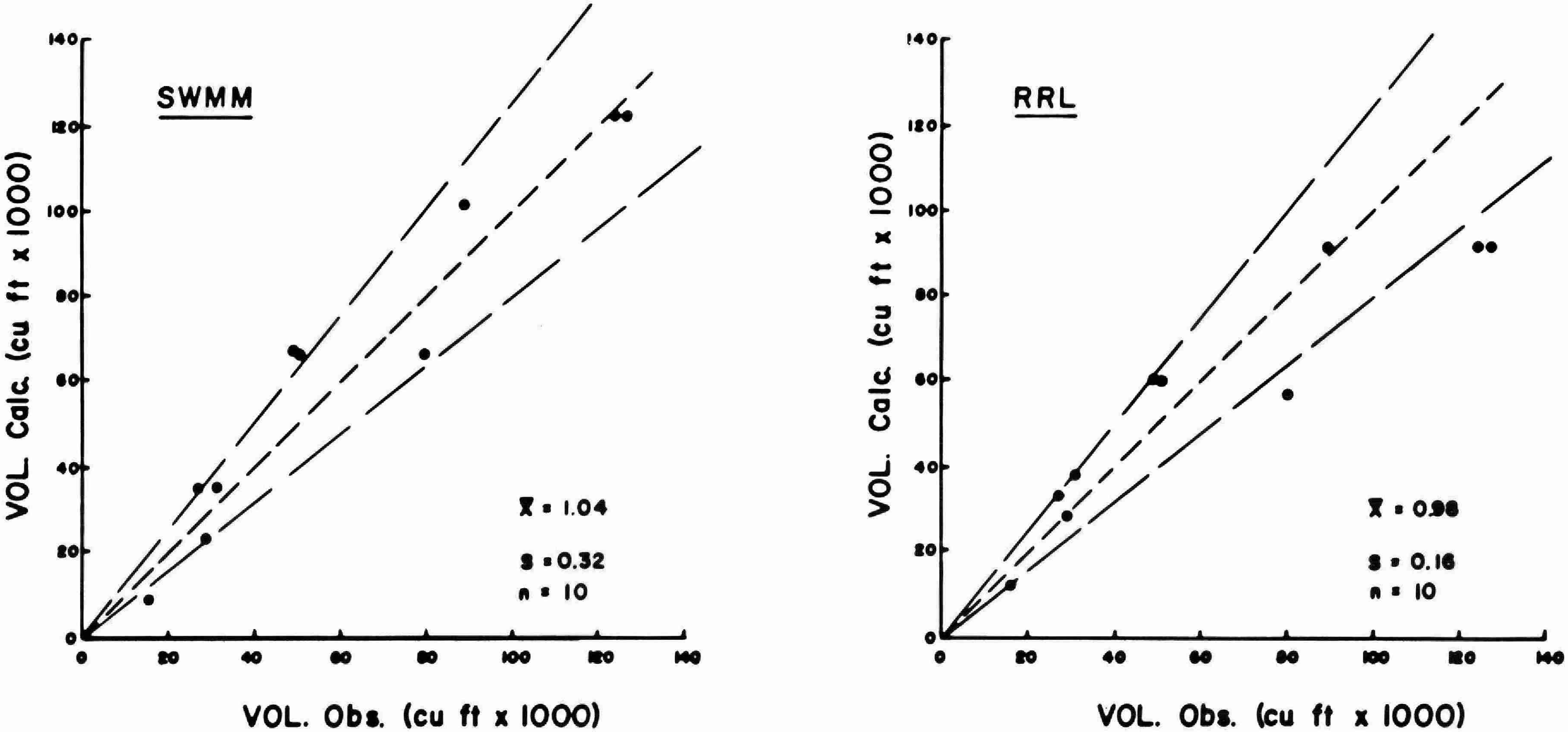
Theoretical line

Lines of ± 20% error

GRAY HAVEN, BALTIMORE

Tp CALCULATED vs. Tp OBSERVED
SWMM AND RRL MODELS

Figure 20



Vol. = total runoff volume

$\bar{X}$ = mean of $\frac{\text{Vol (calc)}}{\text{Vol (obs)}}$

n = no. of observations

Vol (obs) = Vol (calc) — — — — —

LINES OF ± 20% ERROR ——— ———

GRAY HAVEN, BALTIMORE

VOL. CALCULATED vs. VOL. OBSERVED

SWMM AND RRL MODELS

FIG. 21

TABLE 9. RECORDED PEAK FLOWS FOR GRAY HAVEN AND COMPUTED PEAK FLOWS OF THE SWMM, RRL AND MIT MODELS

No.	STORM DATE	MEAS. Qp 1 (cfs)	SWMM Qp 2 (cfs)	SWMM 2/1	RRL Qp 3 (cfs)	RRL 3/1	MIT Qp 4 (cfs)	MIT 4/1
1	6/5/63	80.7	78.0	0.97	59.4	0.74	-	-
2	6/10/63	79.0	78.1	0.99	58.9	0.75	82.5	1.04
3	6/14/63-1	30.8	31.4	1.02	32.9	1.07	34.5	1.12
	-2	23.2	33.5	1.44	27.0	1.16	21.0	0.91
4	6/20/63-1	29.6	34.1	1.15	35.7	1.21	-	-
	-2	22.6	33.7	1.49	26.9	1.19	-	-
5	6/29/63	27.2	30.9	1.13	35.4	1.30	-	-
6	8/1/63-1	88.1	63.3	0.72	59.2	0.67	83.0	0.93
7	8/14/63-1	34.7	38.5	1.11	40.5	1.17	-	-
	-2	16.8	21.6	1.29	19.7	1.17	-	-
	-3	17.1	19.1	1.12	22.6	1.32	-	-
8	8/18/64	19.6	16.1	0.82	23.8	1.22	-	-
9	8/2/65	30.3	34.7	1.15	36.6	1.21	-	-
10	8/12/66-2	19.3	15.1	0.78	15.7	0.81	-	-
11	7/23/65	1.5	-	-	-	-	1.65	1.10
12	7/16/65	5.1	-	-	-	-	7.0	1.37
$\bar{X}$ = MEAN $\frac{\text{Qp calc}}{\text{Qp meas}}$				1.18		1.14		-
S = STANDARD DEVIATION of Qp ratios				0.22		0.22		-

TABLE 10. RECORDED TIMES TO PEAK FOR GRAY HAVEN AND COMPUTED TIMES TO PEAK FOR THE SWMM, RRL AND MIT MODELS

No.	STORM DATE	MEAS. Tp 1 (mins)	SWMM Tp 2 (mins)	SWMM 2/1	RRL Tp 3 (mins)	RRL 3/1	MIT Tp 4 (mins)	MIT 4/1
1	6/5/63	41	39	.95	34	.83	-	-
2	6/10/63	41	39	.95	34	.83	34	.83
3	6/14/63-1	22	21	.96	22	1.00	23	1.05
	-2	44	43	.98	44	1.00	44	1.00
4	6/20/63-1	22	21	.96	22	1.00	-	-
	-2	44	43	.98	44	1.00	-	-
5	6/29/63	28	29	1.04	26	.93	-	-
6	8/1/63-1	12	13	1.08	13	1.08	14	1.17
7	8/14/63-1	16	15	.94	16	1.00	-	-
	-2	48	47	.98	47	0.98	-	-
	-3	110	109	.99	88	0.80	-	-
8	8/18/64	14	12	.86	12	0.86	-	-
9	8/2/65	35	34	.97	35	1.00	-	-
10	8/12/66-2	24	25	1.04	26	1.08	-	-
11	7/23/65	14	-	-	-	-	12	.86
12	7/16/65	11	-	-	-	-	10	.91
$\bar{X}$ = MEAN $\frac{\text{Tp calc}}{\text{Tp meas}}$				0.98		0.89		-
S = STANDARD DEVIATION of Tp ratios				0.05		0.30		-

TABLE 11. RECORDED VOLUMES OF RUNOFF FOR GRAY HAVEN COMPARED TO COMPUTED VOLUMES OF THE SWMM AND RRL MODELS

		MEAS.	SWMM		RRL	
No.	STORM DATE	Vol 1 (cu ft)	Vol 2 (cu ft)	$^{2}/_{1}$	Vol 3 (cu ft)	$^{3}/_{1}$
1	6/5/63	127404	122401	.96	91786	.72
2	6/10/63	123903	122091	.99	91597	.74
3	6/14/63	51231	66249	1.29	59556	1.16
4	6/20/63	48971	67722	1.38	60367	1.23
5	6/29/63	26703	34627	1.30	33204	1.24
6	8/1/63-1	80319	66350	.83	56990	.71
7	8/14/63	88913	101501	1.14	90680	1.02
8	8/18/64	16257	9655	.59	12582	.77
9	8/2/65	31339	35740	1.14	38626	1.23
10	8/12/66	29099	23722	.82	28076	.96
$\bar{X}$ = MEAN $\frac{\text{Vol comp}}{\text{Vol meas}}$				1.04		.98
S = STANDARD DEVIATION of Vol. ratios				.32		.30

TABLE 12. STATISTICAL COMPARISON OF RECORDED RUNOFF HYDROGRAPHS ON THE GRAY HAVEN TEST AREA WITH THE SWMM, AND RRL COMPUTED HYDROGRAPHS

No.	STORM DATE	PARA-METER	SWMM	RRL
1	6/5/63	R	.983	.931
		RS	.979	.866
		ISE	2.29	5.92
2	6/10/63	R	.984	.921
		RS	.981	.864
		ISE	2.23	6.03
3	6/14/63	R	.967	.890
		RS	.907	.871
		ISE	4.59	5.40
4	6/20/63	R	.975	.895
		RS	.865	.842
		ISE	5.57	6.02
5	6/29/63	R	.992	.951
		RS	.912	.834
		ISE	5.97	8.22
6	8/1/63	R	.981	.957
		RS	.934	.890
		ISE	5.43	7.04
7	8/14/63	R	.980	.898
		RS	.962	.871
		ISE	2.05	3.18
8	8/18/64	R	.834	.522
		RS	.745	.444
		ISE	11.46	16.93
9	8/2/65	R	.950	.882
		RS	.936	.845
		ISE	4.39	6.85
10	8/12/66	R	.884	.844
		RS	.933	.953
		ISE	4.14	3.45

Several hydrographs showing typical results for the Gray Haven area are given in Figures 22 and 23.

3.3.4 Comparative study for the Calvin Park test area (Kingston)

The last test area used in the study was the Calvin Park area of Kingston (Figure 24). This test catchment is an 89.4 acre suburban residential development in Kingston, Ontario, typical of most recent subdivisions. It consists primarily of single family units. However, there is also a school, a church, a senior citizens' home and some low rise apartment units. Twenty-seven per cent of the area is impervious, which is lower than the other test areas previously considered, and the roof leaders of the houses are not connected to the storm sewers.

The rainfall and runoff are recorded on the same strip-chart recorder located behind a weir in a manhole at the 42-inch outfall.

The measurements were conducted in 1973 and 1974 by Queen's University and will probably continue during 1975. Ten storms were available for this study. The maximum average intensities for the time of concentration of 15 min ranged from 0.6 to 2.0 in/hr.

Computer runs were made for all the available storms using the SWMM, RRL and UCUR models. Typical comparisons between measured and computed hydrographs are presented in Figures 31 and 32. All the measured peak flow values, times to peak and runoff volumes are compared with the computed values in Figures 26 through 31 and Tables 13 through 15, while the hydrographs are compared statistically in Table 16.

Inspection of Figures 27 and 28 shows that the response for the time to peak is again good for all three models. The response for the peak flow is about the same for the SWMM and UCUR models while the RRL model is somewhat higher (Figures 25 and 26). This is also evident from the following tabulation:

Results of Model Tests on Calvin Park

	MODEL		
	SWMM	UCUR	RRL
Mean $\frac{Qp \text{ computed}}{Qp \text{ measured}}$	1.20	1.23	1.30

Results of Model Tests on Clavin Park (cont'd)	MODEL		
	SWMM	UCUR	RRL
Standard deviation of QP ratios	.217	.248	.155
Mean $\frac{\text{Tp computed}}{\text{Tp measured}}$	.846	.840	1.00
Standard deviation of Tp ratios	.105	.150	.215
Number of Observations	13	13	12
Mean $\frac{\text{Vol computed}}{\text{Vol measured}}$	.833	.830	1.25
Standard deviation of Vol ratios	.138	.142	.235
Number of Observations	10	9	10

The analysis of the results indicates again that for the rainfalls considered, the contribution of the pervious areas is negligible. Under these conditions the RRL method can give higher peaks than those measured because, as indicated in Subsection 3.2.1, the model does not consider surface storage.

3.4 Assessment of the SWMM, UCUR, and RRL Models

3.4.1 Conclusions from the comparative simulations

The one-event simulations described in Section 3.3 lead to the following conclusions:

a) Simulations with a large number of storms indicate discrepancies between the predicted and measured peak flows, which are more significant than shown in the publications of the different model builders. This is evidenced in the following table which gives the number of predictions with an error greater than an arbitrary limit of $\pm$ 20 per cent.

   Although the errors in prediction of the peak flow are somewhat larger than indicated in some of the previously published results, these results obtained without any optimization or additional calibration are considered

acceptable if compared with the Rational Method (see Section 3.7), the general state of the art of hydrologic modelling, and the needs of the practice.

Test Area	Number of Peaks Considered *	Number of Peak Flow Predictions with Errors Greater than ± 20% — MODEL: SWMM	UCUR	RRL
Oakdale	17	8	5 **	7
Gray Haven	14	5	-	8
Calvin Park	14	10	8	9

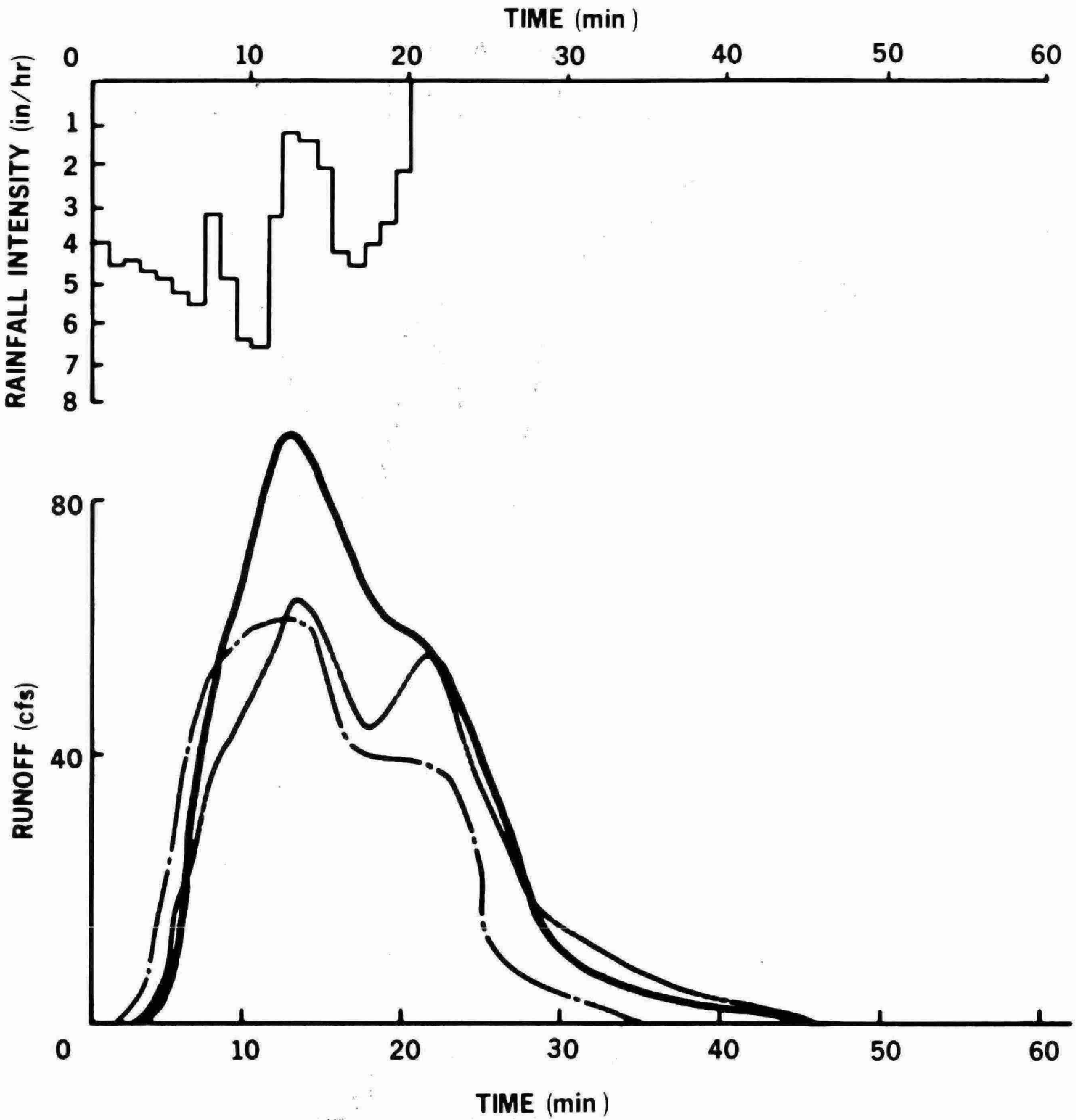
* Some recorded storms have 2 distinct peaks.

** Only 14 peaks considered.

b) The mean values of $\frac{\text{Q peak computed}}{\text{Q peak recorded}}$ and $\frac{\text{T peak computed}}{\text{T peak recorded}}$ do not indicate a clear cut advantage for any of the three models. The SWMM model results are only marginally better according to the statistical analysis. This, of course, is valid only for the specific conditions of the three test areas and rainfalls considered. The areas, however, encompass a percentage of imperviousness of 27 to 68 per cent and their sizes range from 13 to 90 acres. Storms were measured for periods ranging from 2 years to 5 years. The results are, therefore, typical of what can be expected from the models when applied to small areas.

The average value of $\frac{\text{Q peak computed}}{\text{Q peak recorded}}$ is larger than unity for the SWMM and UCUR models on all three test areas, while for the RRL model it is less than unity at one area and larger than one on the two others. This inconsistency of the RRL method will be further discussed in another section.

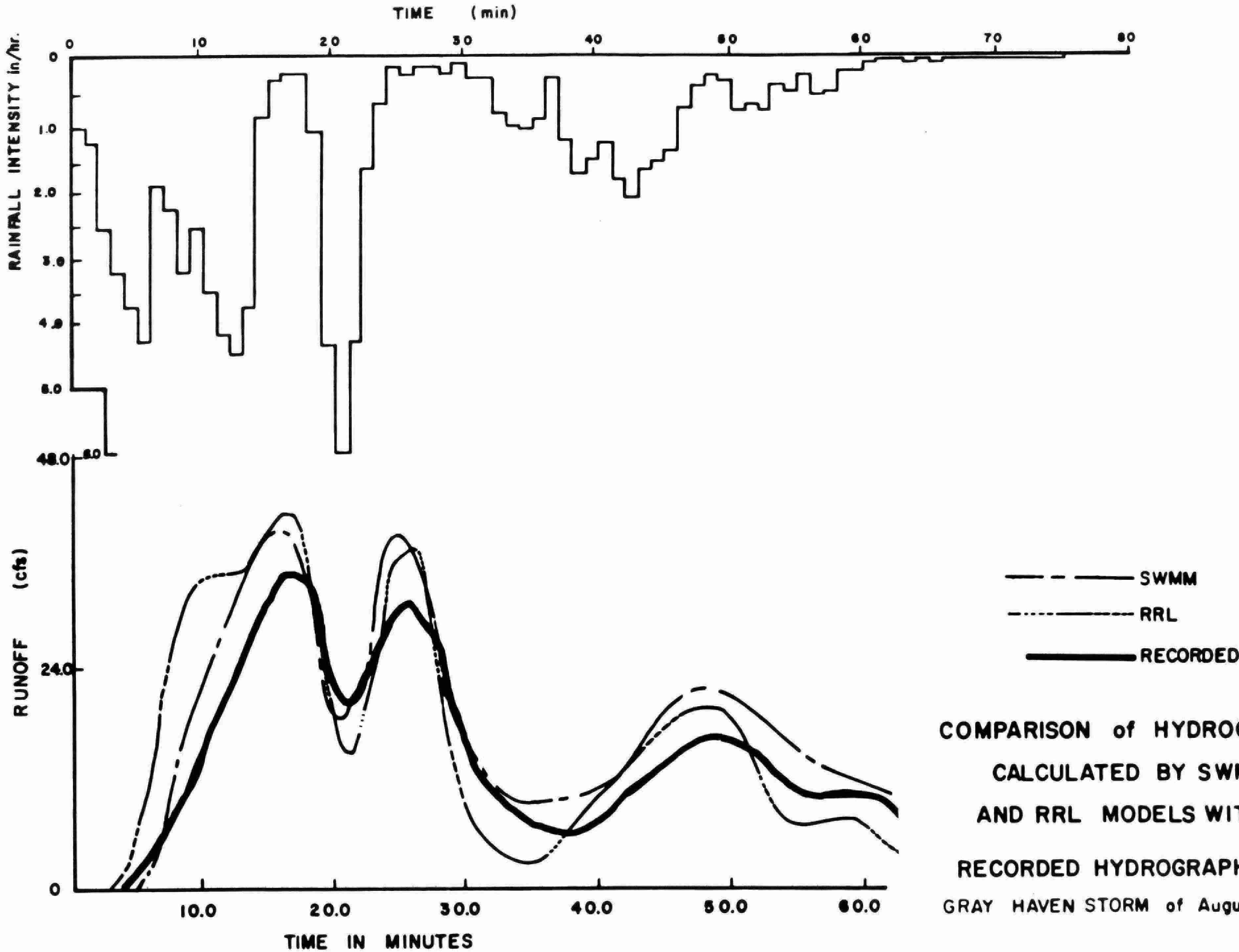
The prediction of the time to peak is better than that of the peak flow for all three methods.



COMPARISON OF HYDROGRAPHS CALCULATED BY SWMM AND RRL MODELS WITH RECORDED HYDROGRAPH

GRAY HAVEN STORM OF AUGUST 1, 1963

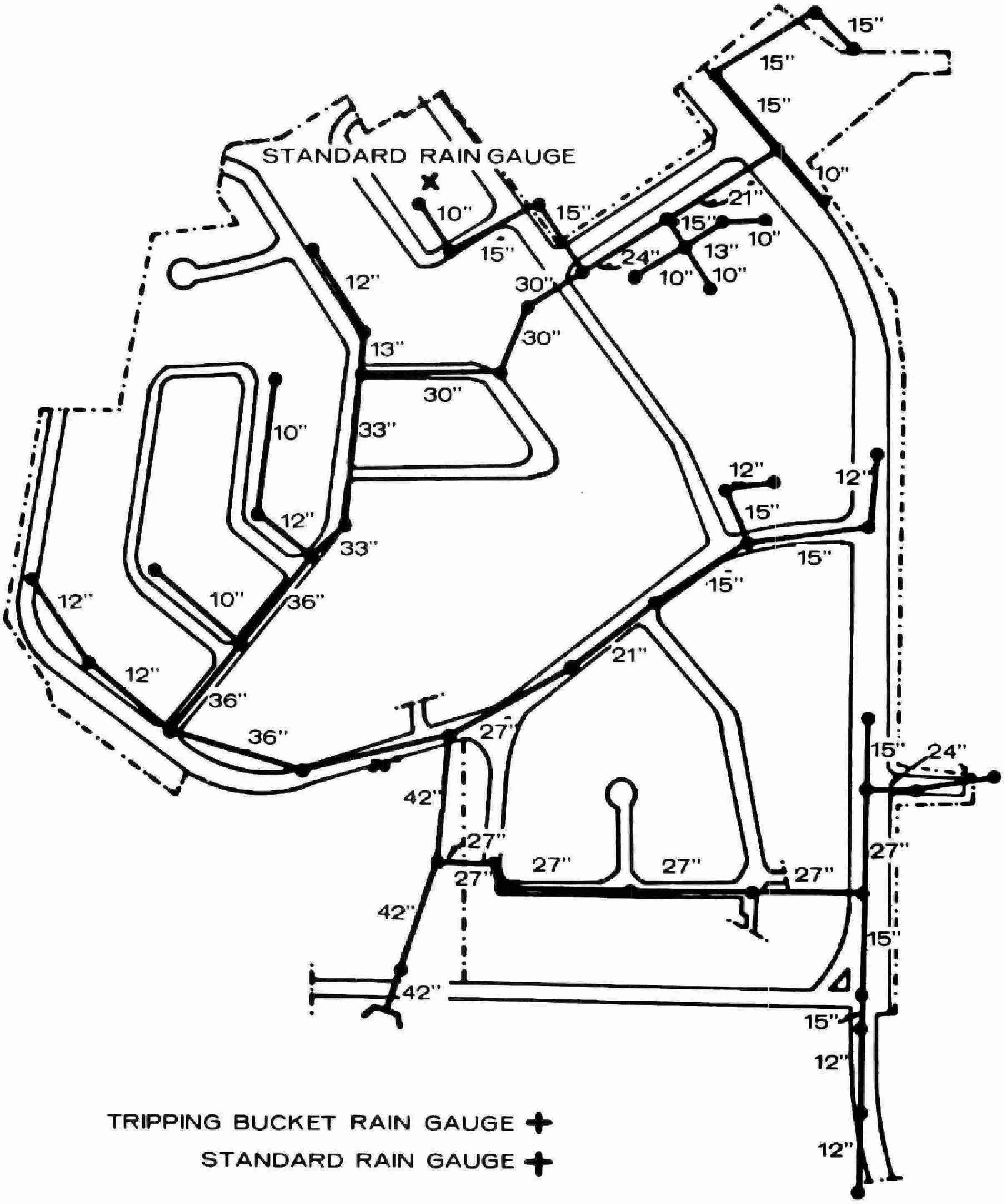
Figure 22



COMPARISON of HYDROGRAPHS CALCULATED BY SWMM AND RRL MODELS WITH RECORDED HYDROGRAPH

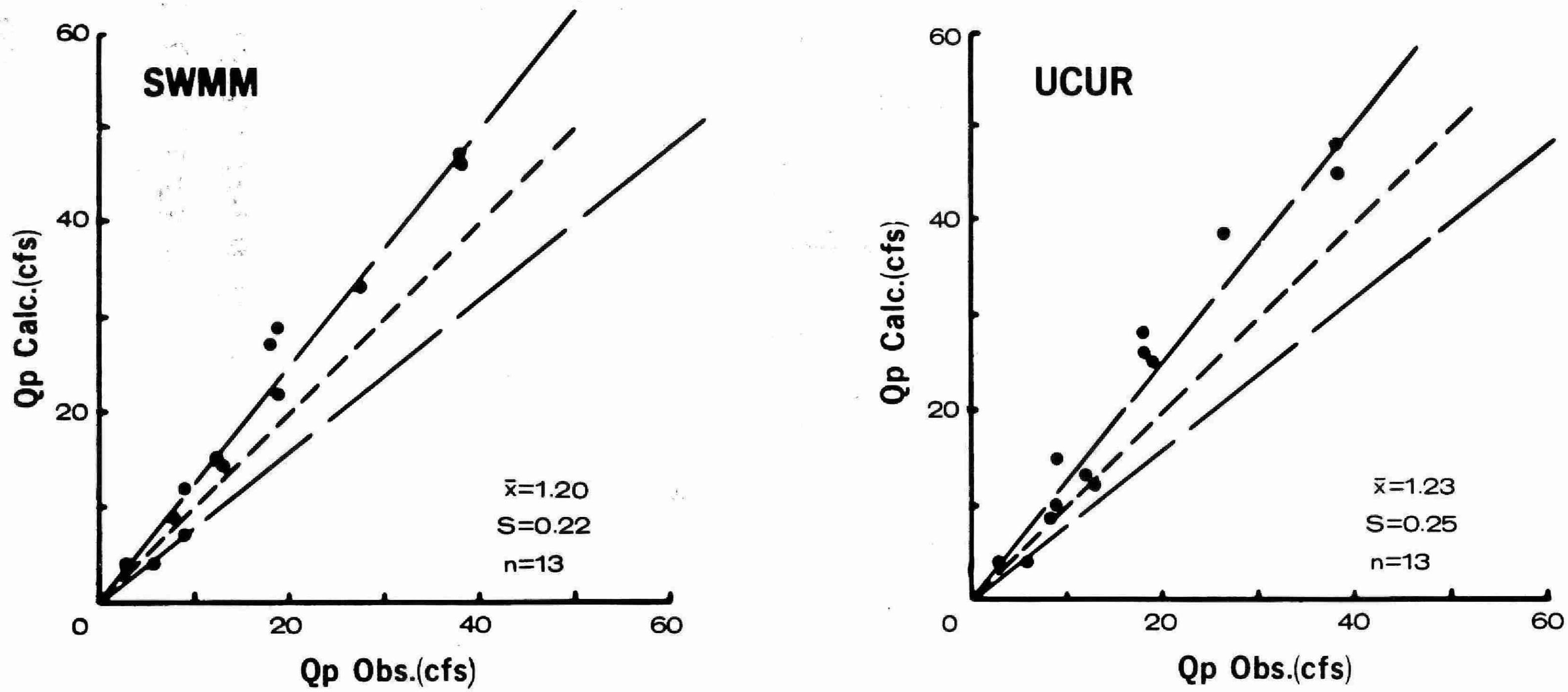
GRAY HAVEN STORM of August 14, 1963

FIG. 23



CALVIN PARK TEST AREA
KINGSTON, ONTARIO

Figure 24



Qp=peak flow

x̄=mean of $\frac{Qp(calc)}{Qp(obs)}$

n=no. of observations

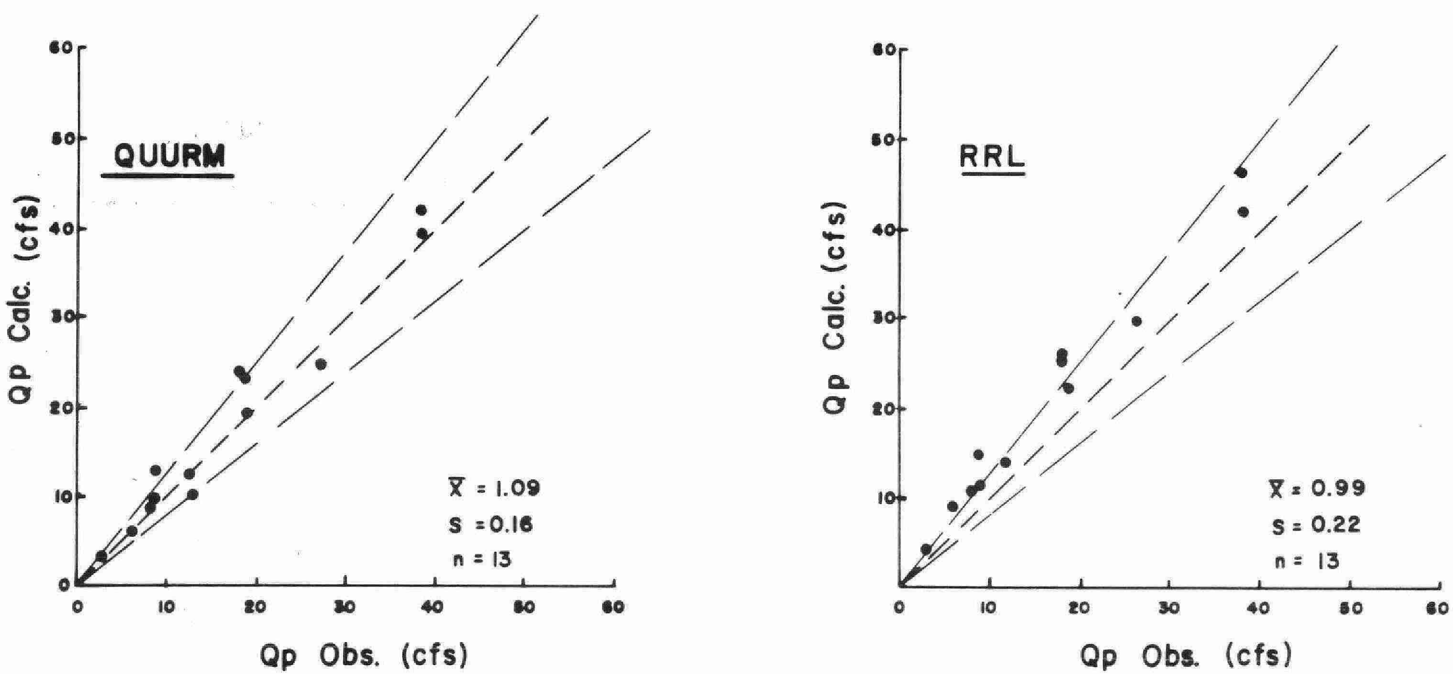
Qp(obs)=Qp(calc) – – – – –

Lines of ± 20% error —— ——

CALVIN PARK, KINGSTON

Qp CALCULATED vs. Qp OBSERVED SWMM AND UCUR MODELS

Figure 25



Qp = peak flow

x = mean of $\frac{Qp\,(calc)}{Qp\,(obs)}$

n = no. of observations

Qp (obs) = Qp (calc) — — — — —

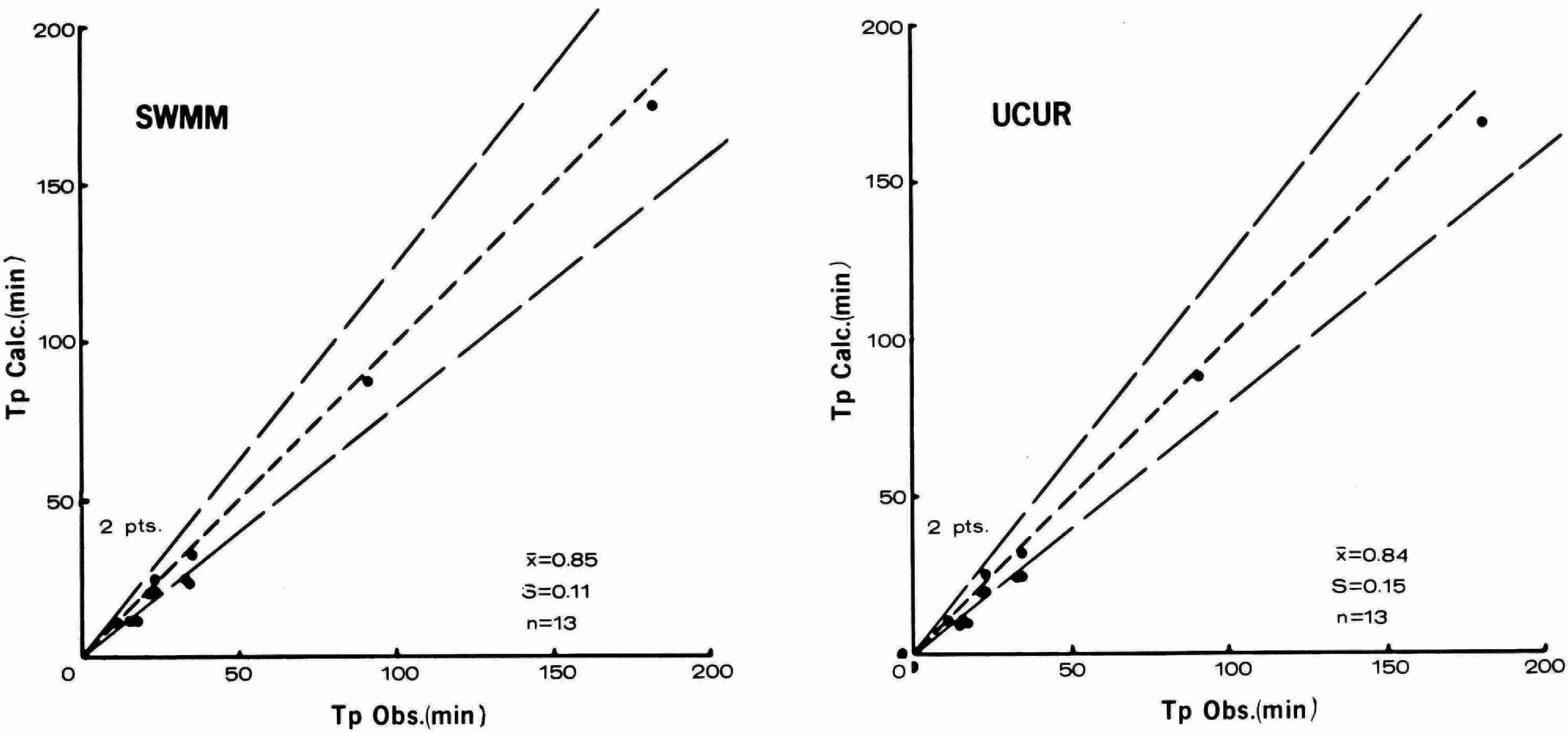
LINES OF ± 20% ERROR —— ——

CALVIN PARK, KINGSTON

Qp CALCULATED vs. Qp OBSERVED

QUURM AND RRL MODELS

FIG. 26



Tp=time to peak

$\bar{x}$=mean of $\frac{Tp(calc)}{Tp(obs)}$

n=no. of observations

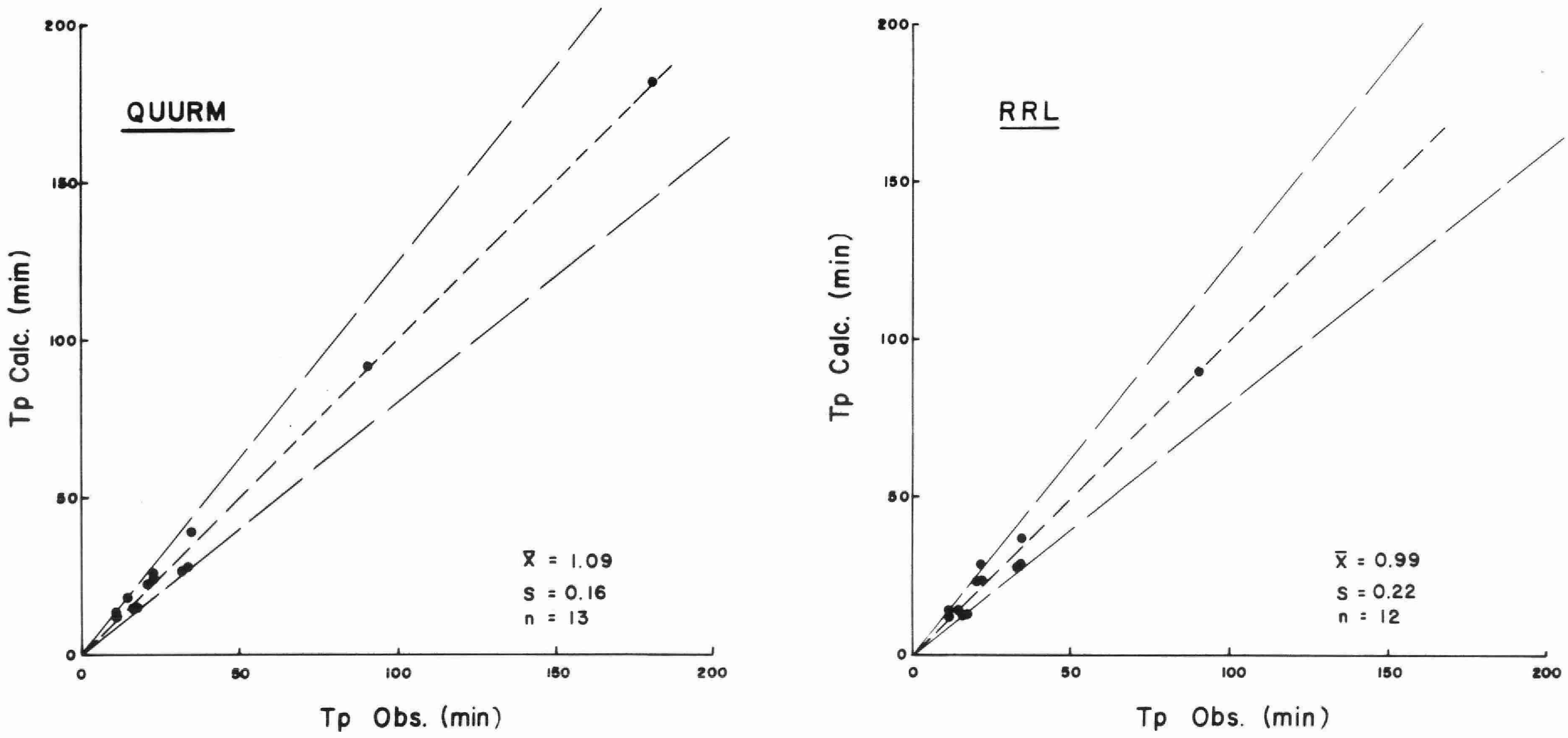
Tp(obs)=Tp(calc) – – – – –

Lines of ± 20% error —— ——

CALVIN PARK, KINGSTON

Tp CALCULATED vs. Tp OBSERVED
SWMM AND UCUR MODELS

Figure 27



Tp = time to peak

$\bar{X}$ = mean of $\frac{\text{Tp (calc)}}{\text{Tp (obs)}}$

n = no. of observations

Tp (obs) = Tp (calc) – – – – –

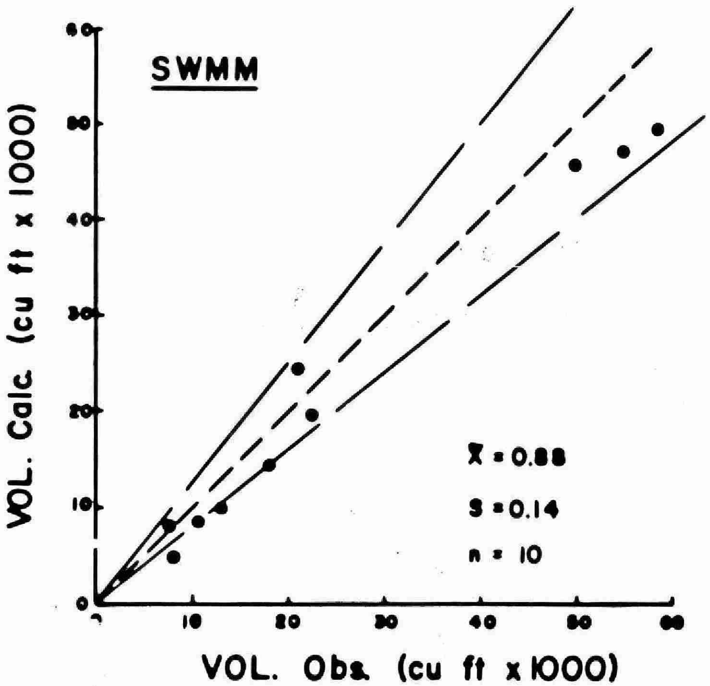
LINES OF ± 20% ERROR ——— ———

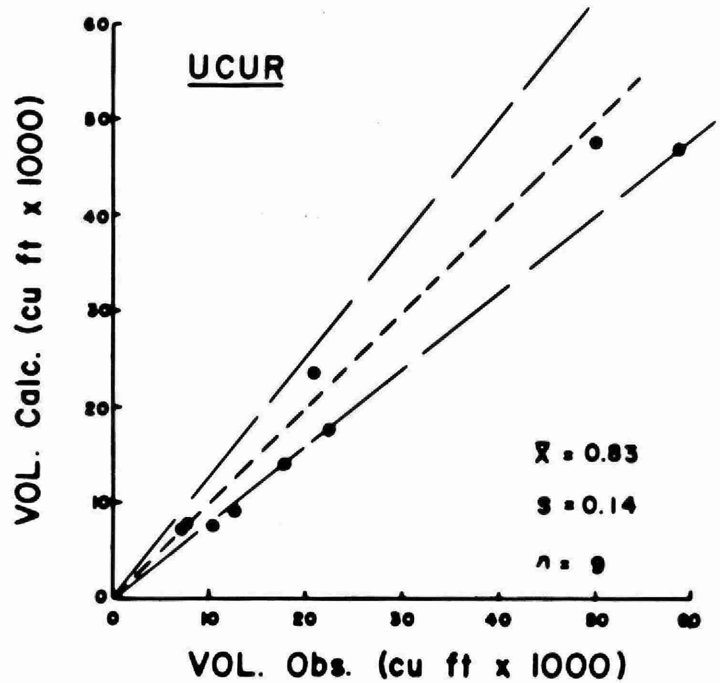
CALVIN PARK, KINGSTON

Tp CALCULATED vs. Tp OBSERVED

QUURM AND RRL MODELS

FIG. 28





Vol = total runoff volume

$\bar{X}$ = mean of $\frac{Tp(calc)}{Tp(obs)}$

n = no. of observations

Vol (obs) = Vol (calc) – – – – –

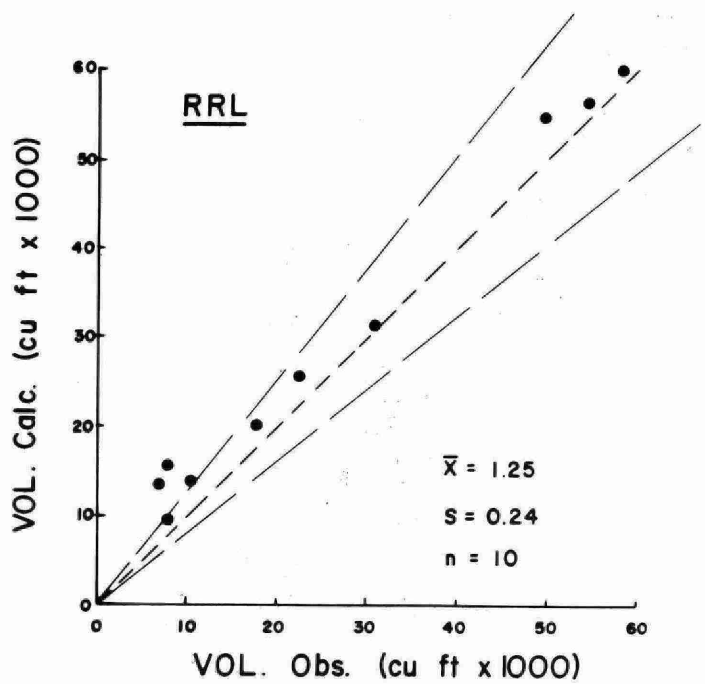
LINES OF ±20 % ERROR —— ——

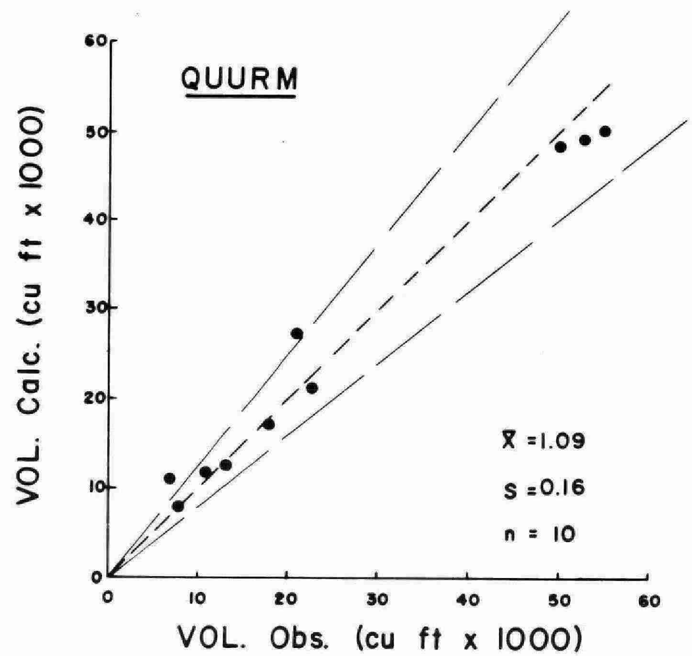
CALVIN PARK, KINGSTON

VOL. CALCULATED vs. VOL. OBSERVED

SWMM AND UCUR MODELS

FIG. 29





Vol = total runoff volume

$\bar{x}$ = mean of $\frac{\text{Vol.(calc)}}{\text{Vol (obs)}}$

n = no. of observations

Vol(obs) = Vol (calc) – – – – –

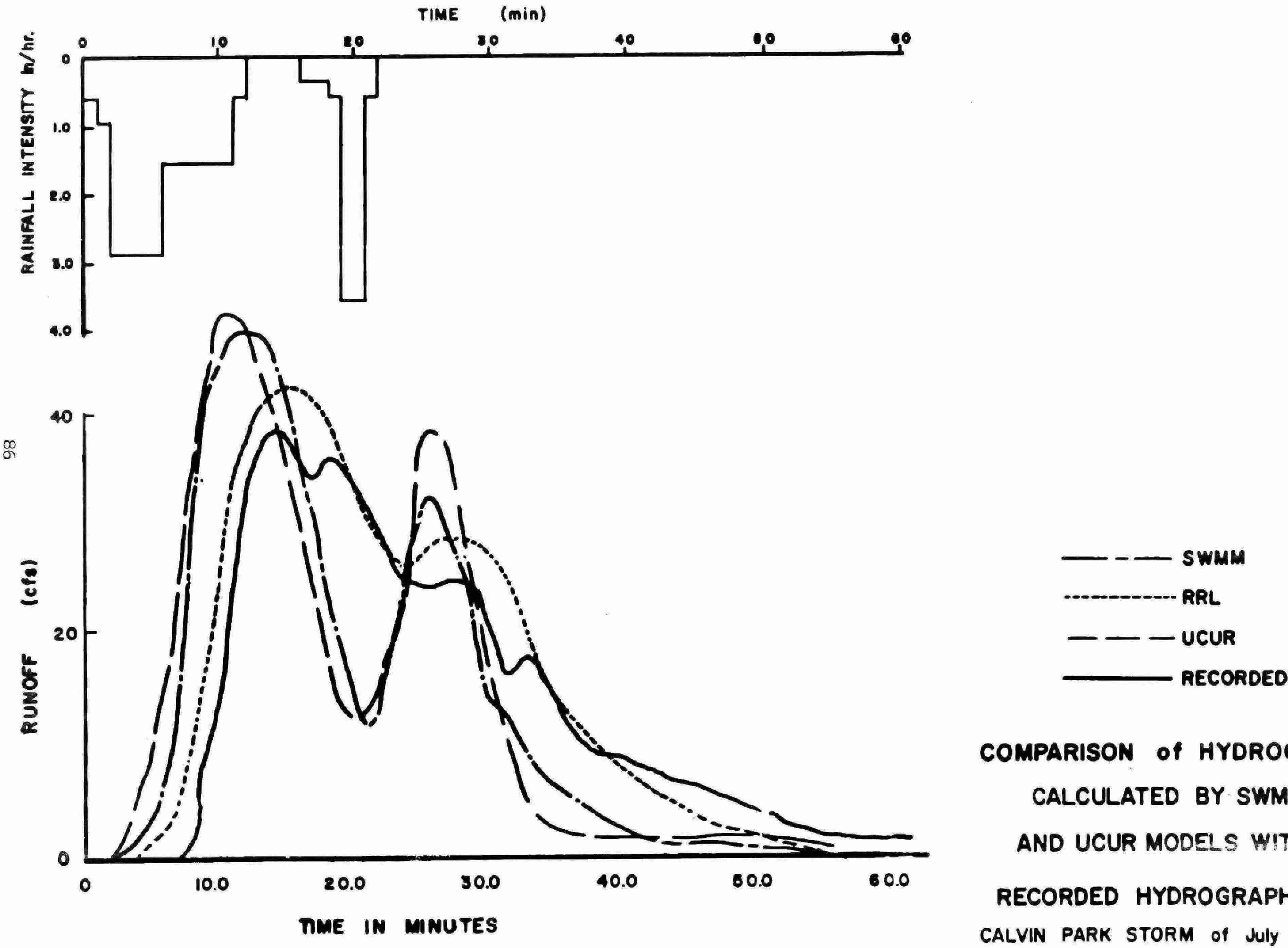
LINES OF ±20% ERROR —— ——

CALVIN PARK, KINGSTON

VOL. CALCULATED vs. VOL. OBSERVED

RRL AND QUURM MODELS

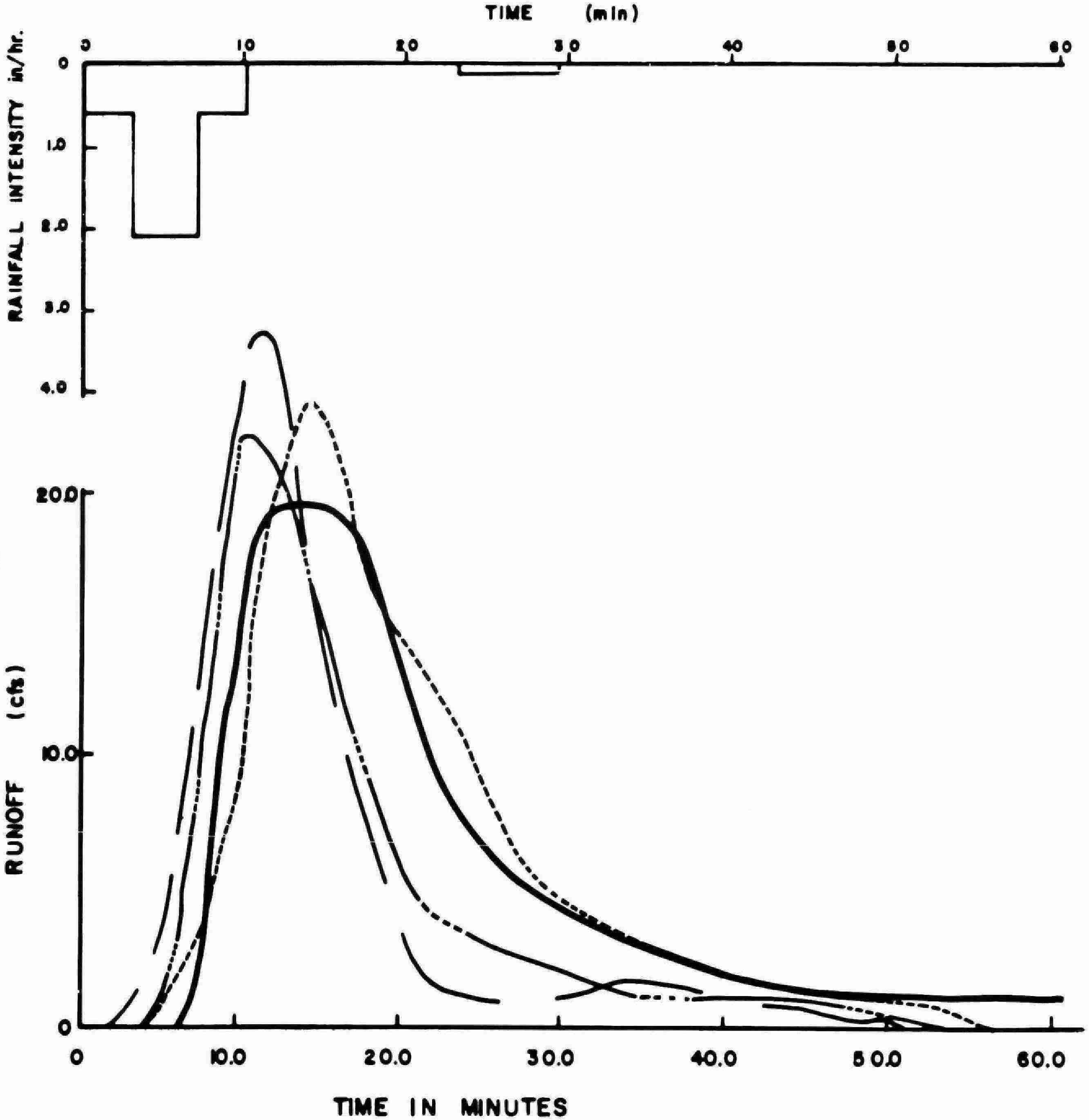
FIG. 30



COMPARISON of HYDROGRAPHS CALCULATED BY SWMM, RRL AND UCUR MODELS WITH RECORDED HYDROGRAPH

CALVIN PARK STORM of July 25, 1972

FIG. 31



COMPARISON of HYDROGRAPHS CALCULATED BY SWMM, RRL AND UCUR MODELS WITH RECORDED HYDROGRAPH

CALVIN PARK STORM of August 23, 1972

FIG. 32

TABLE 13. RECORDED PEAK FLOWS FOR CALVIN PARK AND COMPUTED PEAK FLOWS OF THE SWMM, RRL AND UCUR MODELS

No.	STORM DATE	MEAS. Qp 1 (cfs)	SWMM Qp 2 (cfs)	SWMM 2/1	RRL Qp 3 (cfs)	RRL 3/1	UCUR Qp 4 (cfs)	UCUR 4/1
1	7/25/72-1	38.1	46.0	1.21	42.2	1.11	48.0	1.26
	-2	26.6	32.7	1.22	28.9	1.09	38.6	1.45
3	8/9/72-1	8.2	9.1	1.11	10.9	1.33	8.6	1.05
4	8/9/72-2	18.7	28.6	1.53	25.3	1.35	28.6	1.53
5	8/14/72-1	12.3	15.7	1.28	14.1	1.14	13.7	1.11
	-2	13.2	14.8	1.12	-	-	12.5	.95
6	8/18/72-1	6.0	4.4	0.73	9.2	1.53	4.3	.72
	-2	3.4	4.7	1.38	4.8	1.42	4.2	1.24
7	8/23/72	19.0	22.0	1.16	22.8	1.20	25.7	1.35
8	6/1/73	9.4	7.0	0.75	11.7	1.25	10.9	1.06
9	7/12/73	9.0	12.2	1.36	15.1	1.67	15.1	1.68
10	7/26/73	18.7	27.8	1.48	25.7	1.37	26.2	1.40
11	8/1/73	38.6	47.2	1.22	46.7	1.21	45.6	1.18
$\bar{X}$ = MEAN $\frac{\text{Qp calc}}{\text{Qp meas}}$				1.20		1.31		1.23
S = STANDARD DEVIATION of Qp ratios				0.22		0.16		0.25

TABLE 14. RECORDED TIMES TO PEAK FOR CALVIN PARK AND COMPUTED TIMES TO PEAK OF THE SWMM, RRL AND UCUR MODELS

No.	STORM DATE	MEAS. Tp 1 (mins)	SWMM Tp 2 (mins)	SWMM 2/1	RRL Tp 3 (mins)	RRL 3/1	UCUR Tp 4 (mins)	UCUR 4/1
1	7/25/72-1	12	10	.77	15	1.25	11	.92
	-2	23	24	1.04	28	1.22	25	1.09
3	8/9/72-1	23	21	.91	24	1.04	20	.87
4	8/9/72-2	22	20	.91	24	1.09	20	.91
5	8/14/72-1	91	87	.96	90	.99	88	.97
	-2	181	176	.97	-	-	178	.98
6	8/18/72-1	15	12	.80	14	.93	9	.60
	-2	35	33	.94	37	1.06	32	.91
7	8/23/72	12	11	.92	14	1.17	11	.92
8	6/1/73	17	12	.71	14	.82	10	.59
9	7/12/73	16	12	.75	14	.88	11	.69
10	7/26/73	33	24	.73	28	.85	24	.73
11	8/1/73	34	23	.68	29	.85	24	.71
$\bar{X}$	= MEAN $\frac{\text{Tp calc}}{\text{Tp meas}}$			.85		.99		.84
S	= STANDARD DEVIATION of Tp ratios			.11		.22		.15

TABLE 15. RECORDED RUNOFF VOLUMES FOP CALVIN PARK AND COMPUTED RUNOFF VOLUMES OF THE SWMM, RRL AND UCUR MODELS

		MEAS.	SWMM		RRL		UCUR	
No.	STORM DATE	Vol 1 (cu ft)	Vol 2 (cu ft)	2/1	Vol 3 (cu ft)	3/1	Vol 4 (cu ft)	4/1
1	7/25/72	50016	45986	0.91	54749	1.09	47782	.96
3	8/9/72-1	13002	10221	0.78	15721	1.21	9019	.69
4	8/9/72-2	22764	19320	0.85	25705	1.13	17802	.78
5	8/14/72	55104	47205	0.85	56139	1.02	-	-
6	8/18/72	10680	8670	0.81	13957	1.31	7689	.72
7	8/23/72	18210	14608	0.81	20432	1.12	14165	.78
8	6/1/73	7974	5333	0.67	10352	1.30	4796	.60
9	7/12/73	7530	8391	1.10	13701	1.82	7627	1.01
10	7/26/73	21260	24593	1.16	31428	1.48	23834	1.12
11	8/1/73	58500	49430	0.84	59410	1.02	47250	.81
$\bar{X}$ = MEAN $\frac{\text{Vol calc}}{\text{Vol meas}}$				.88		1.25		0.83
S = STANDARD DEVIATION of Vol ratios				.14		.24		.14

TABLE 16. STATISTICAL COMPARISON ON RECORDED RUNOFF HYDROGRAPHS ON THE CALVIN PARK TEST AREA WITH THE SWMM, RRL AND UCUR COMPUTED HYDROGRAPHS

No.	STORM DATE	PARA-METER	SWMM	RRL	UCUR
1		R	.826	.983	.677
	7/25/72	RS	.776	.966	.616
		ISE	8.17	3.14	10.87
3		R	.878	.917	.648
	8/9/72	RS	.859	.829	.617
		ISE	6.11	6.72	10.51
4		R	.702	.950	.526
	8/9/72	RS	.512	.865	.232
		ISE	12.65	6.63	16.85
5		R	.701	.890	-
	8/14/72	RS	.727	.917	-
		ISE	4.42	2.44	-
6	8/18/72	R	.789	.951	.660
		RS	.815	.759	.688
		ISE	5.67	6.50	7.94
7		R	.858	.989	.704
	8/23/72	RS	.819	.971	.597
		ISE	8.64	3.47	12.87
8		R	.311	.601	-.120
	6/8/73	RS	.335	.357	-.516
		ISE	17.15	16.87	30.74
9		R	.448	.778	-
	7/12/73	RS	.131	.002	-
		ISE	19.89	21.32	-
10		R	.173	.558	.064
	7/26/73	RS	-.363	.114	-.465
		ISE	22.75	18.33	23.57
11		R	.490	.793	.584
	8/1/73	RS	.193	.649	.173
		ISE	13.87	9.13	19.76

3.4.2 Comparison of overland flow hydrographs

The following aspects were considered in the theoretical analysis:

a) As shown in Figure 5, the SWMM model uses the Manning and continuity equations for overland flow. It also has a very simple representation of the depression storage, a constant value being assumed for a given percentage of the area and subtracted from the flow depth.
As indicated in Figure 6, the UCUR model uses for the overland flow an empirical relationship fitted to both experimental data and data obtained by the kinematic wave method. Depression storage is simulated by an exponential relationship (55). An analysis of the program, confirmed by discussion with one of the authors, indicates, however, that there are some simplifications in the infiltration submodel which are not evident in the published paper (56). For example, depression storage cannot be depleted by infiltration when rainfall intensity decreases to a value less than the infiltration capacity. In this case, the program automatically assigns the depression storage as full.
The RRL method considers a runoff coefficient of 100% for the impervious areas; that is, it completely neglects the detention storage. Losses due to depression storage are considered by assuming a constant percentage reduction of the rainfall for each time step instead of a depression depth (82).
Both SWMM and UCUR models use the Horton's equation for infiltration while the latter model has the additional feature of offsetting the infiltration capacity curve to account for situations when initial rainfall intensity is less than the initial infiltration capacity. This advantage is countered by the simplifying assumptions regarding the depletion of depression storage. The RRL

method considers that the infiltration capacity is always higher than rainfall intensity on pervious areas. This was true for most of the measurements in the test catchments, and may be valid in many urban areas, but it is not usually true for design conditions and in general for rainfalls with higher intensities than those used in the previous analyses. This aspect of model application will be discussed further.

Although the approximations of the RRL models make it more simple, the lack of consideration of pervious areas and provision for infiltration leads to inconsistencies.

In some cases it may give higher values than those recorded due to lack of surface storage, while in others it gives lower results because of the disregard of contributions from pervious areas. (See Section 3.)

It follows, therefore, that from a theoretical viewpoint the simulation of overland flow hydrographs by the SWWM model is logically closer to the real phenomena if compared to the RRL and UCUR models although improvements of the other methods are possible, such as:

i) modification of the RRL method to account for possible contribution from pervious areas and surface storage.

ii) modification of the UCUR model to eliminate the assumptions connected with depletion of depression storage.

iii) use of an improved routing routine in the UCUR model.

However, the influence of the approximate routing method used in SWMM needs further consideration.

b) Studies conducted at MIT indicated that a very accurate method for modelling overland flow on a plane is the kinematic wave method (22).

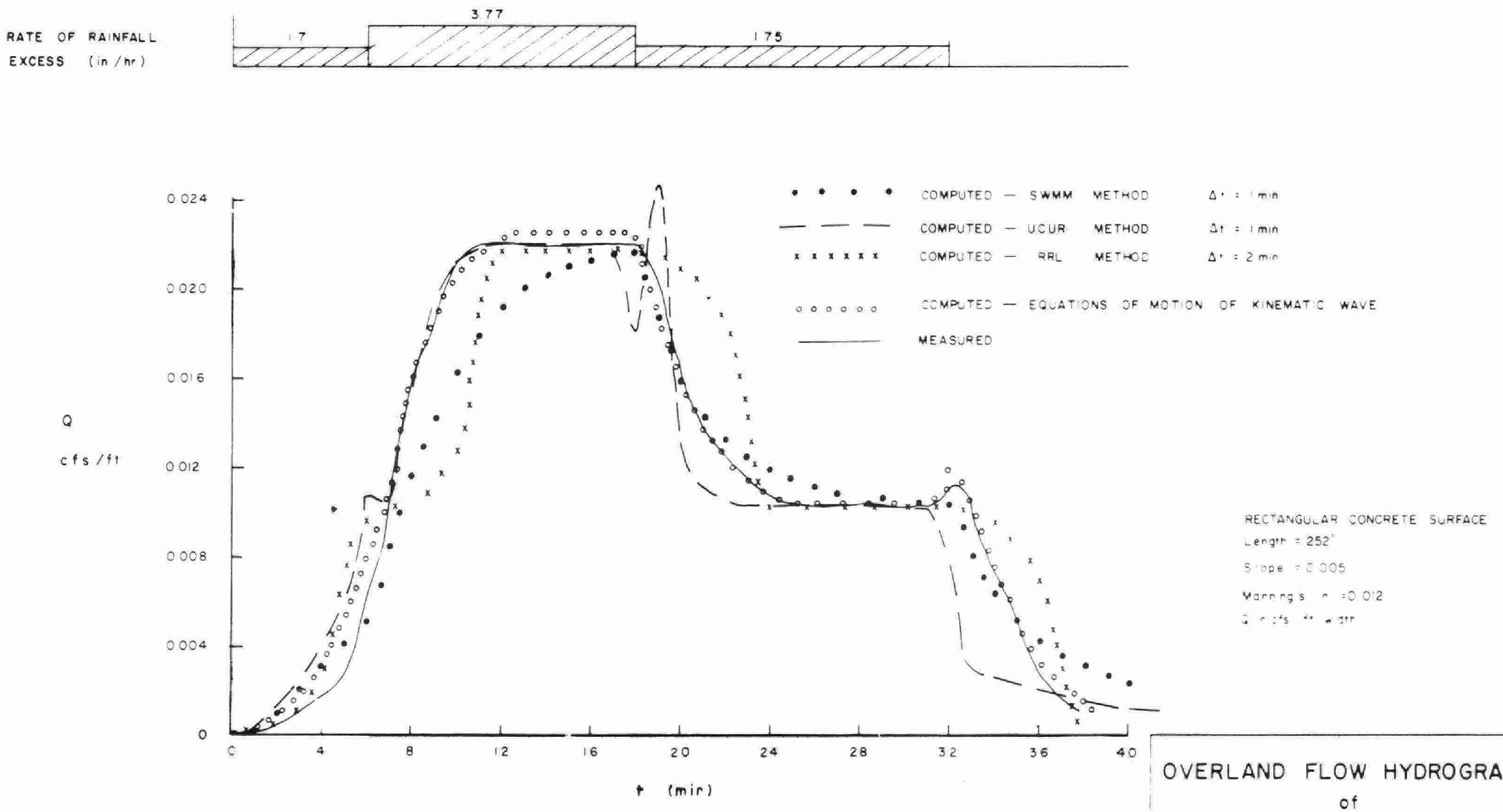
Evidently the integration by this method requires more computer time than the methods of the three analyzed models. Comparisons between the SWMM model and kinematic wave method

have been done by Cheng for planes with different lengths (15). It was found that because the SWMM eliminates the effect of the length of travel on the overland flow depth, the detention storage is too large and some delay may occur in the time of concentration. It follows that the peak rate of flow may be underestimated and the error increases with the length of the plane.

In order to assess the error of the SWMM, a comparison was made between measurements, computations by the kinematic wave method and the SWMM for a 252 foot parking lot. Rainfall, runoff and kinematic wave data were obtained from another study (22). The results presented in Figure 33 indicate that the SWMM gives the same peak value although, as found by Cheng, the detention storage is slightly too large. Comparisons made with the UCUR model on this paved area gave a closer agreement as the model uses an empirical formula based on experimental data. The inconsistencies of the UCUR model indicated above may give differences where infiltration is a factor, however. The length of the equivalent rectangular subareas for the studied test areas varied from 20 ft to 100 ft. For these lengths, the error compared with the kinematic wave method is much smaller and SWMM performs satisfactorily. It follows that the approximation of the SWMM overland routing may be significant only when a large subarea is considered; that is, in the case of very coarse schematizations. In practical applications, where the slope is not always a physical parameter because of the irregularity of an area, one may account for this effect by modifying the default value of the slope or the roughness coefficient. The model has the advantage of simplicity compared to the kinematic wave assumptions.

3.4.3 Routing through conduits

The UCUR model considers the same time-offset method for the



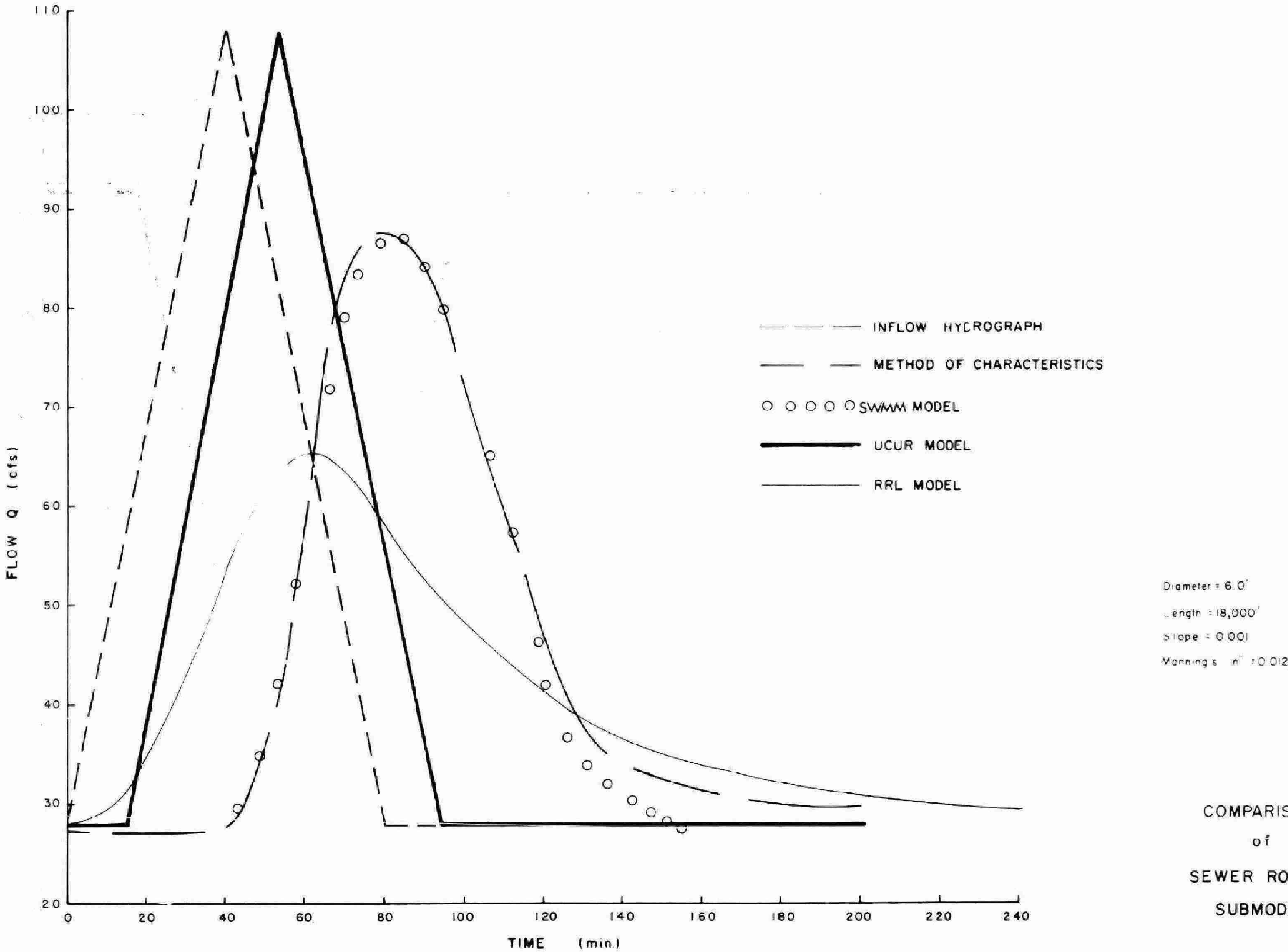
OVERLAND FLOW HYDROGRAPHS
of
SWMM, RRL AND UCUR MODELS
compared to
KINEMATIC WAVE EQUATION AND
RECORDED RESULTS.
FIG. 33

routing in the conduits of the basic subcatchment and for those of the main sewer system. An average velocity for the inflow hydrographs is determined by weighting the velocities in each time increment according to the corresponding inflow volume (see Figure 6). The inflow hydrograph is shifted in time, without changing in shape, a distance equal to the travel time corresponding to this average velocity. For long sewer lengths, this method produces higher peak flows occurring at earlier times than other more exact methods.

The RRL method also uses a simple hydrologic routing technique for all types of pipes (See Figure 9). The conduit is considered to be a reservoir in which inflow equals outflow plus storage and the volume of storage is determined using an assumption of a constant depth of flow throughout the pipe for each time step.

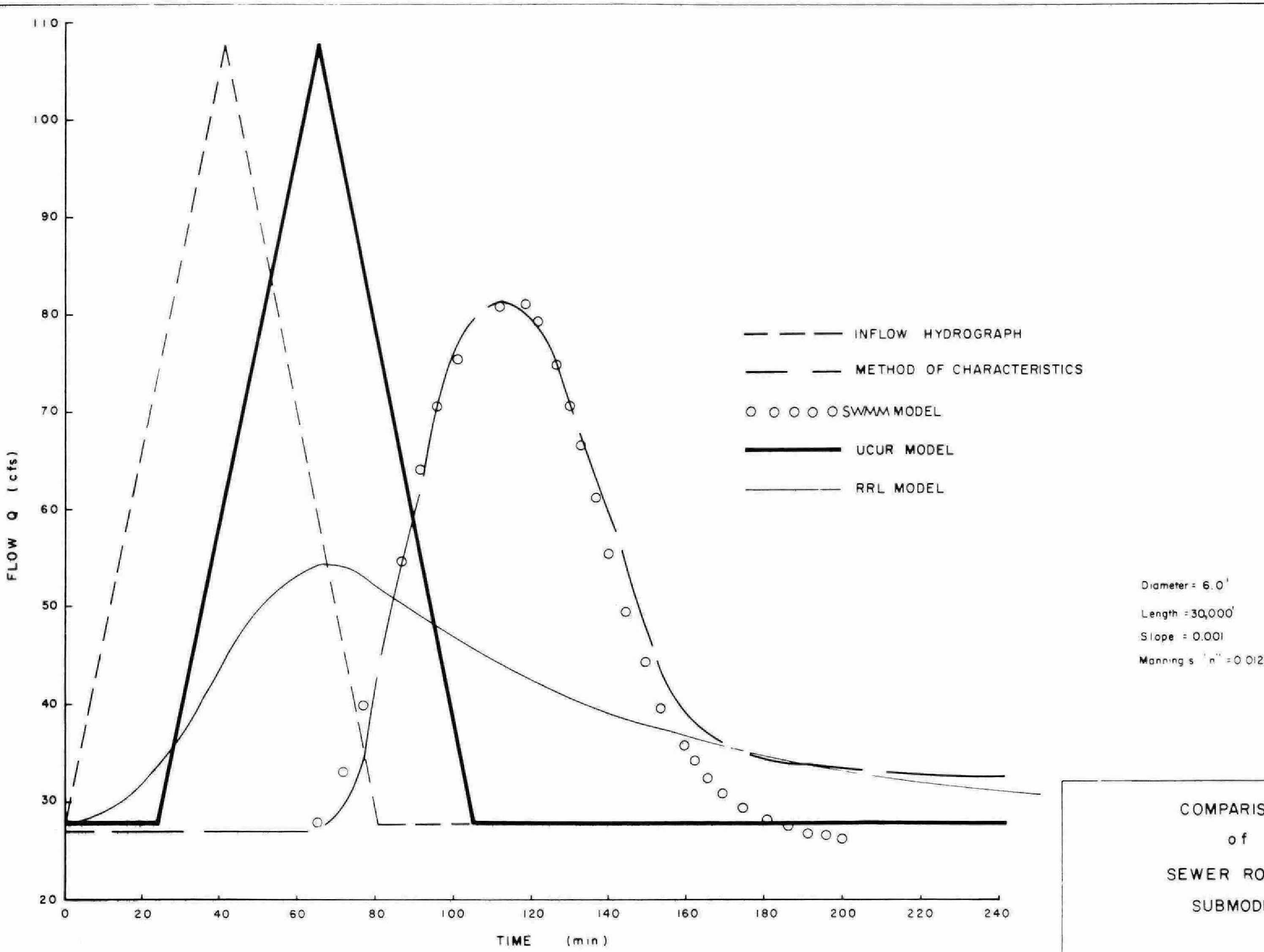
The SWMM uses different routing techniques for the conduits of the subcatchments (called "gutters" in the original publications) and for the main sewer lines. The technique for the gutters is no more sophisticated than either of the previously mentioned methods. For the main sewer lines, however, a special transport routine is used based on the dynamic wave approximation of the St. Vennant equations. This is necessary primarily when the length of the pipes becomes large as indicated in Figures 34 and 35 which compares the results given by the three models and the exact solution by the method of characteristics for conduits of two different lengths.

Another additional feature of the SWMM Transport Routine is that it considers storage in case of surcharge and incorporates internal storage and structures. In the case of surcharge, the inflow in excess of the conduit capacity is stored at the inlet. The downstream flow in the conduit is then maintained at the full flow capacity until this storage is dissipated. The initial transport routine of the SWMM was used for quality and quantity routing. Several changes were made at the University of Florida during this study, however. A very recent improvement is the development of a quantity routing routine that is independent of the quality routing aspects which were combined with quantity routing in the old transport routine. This new routine has



COMPARISON
of
SEWER ROUTING
SUBMODELS

FIG. 34



COMPARISON
of
SEWER ROUTING
SUBMODELS

FIG. 35

been tested and is now operational. An additional sizing subroutine was also recently provided by the University of Florida.

It is considered, therefore, that from the routing point of view the SWMM has the following advantages compared to the RRL and UCUR models:

a) Flexibility by the use of different routines for the basic subcatchments and the main conduit system.

b) Availability of a sophisticated routing program for the main system provided also with a sizing routine. This routine accepts, as input, hydrographs determined by any method.

3.4.4 Cost of modelling

The cost of modelling is determined by the effort in input data collection, program input requirements and computer costs. All of these factors are also dependent on the degree of schematization used.

Because of its simplicity, the RRL model is the least expensive in all aspects of its application, requiring the least amount of subcatchment data of the three models. It is also the only one which can be applied without a computer (82).

Although the SWMM requires much the same data base as the UCUR model, the effort required for input data preparation for the UCUR model is greater than for the SWMM. This is the result of the SWMM's ability to accept average surface characteristics for a non-homogeneous subcatchment while the UCUR requires characteristic homogeneous subcatchment of a specified constant width. All subcatchments must be completely pervious or impervious in the UCUR model as well. This difference really represents a more detailed schematization requirement for the UCUR model.

In the original version of the model, the UCUR computer program required considerably more processing (CPU) time than the SWMM, but subsequent revisions by Monash University in Australia have resulted in drastic improvements in computational efficiency. This version required somewhat less CPU time than the SWMM model for the test areas

of this study. Although this advantage is more significant for very large areas because of the simplified routing procedure of the UCUR, Figure 35 indicates that this simplified routing may be too approximative. The average CPU times on an IBM 370 for the RRL, UCUR and SWMM models are given in Table 17 for each test area. As can be seen from these figures, the computer costs are not large for any of the models when applied to basic subcatchments (less than $10.00) so that it is felt that model selection on this basis is not warranted.

3.4.5 Additional studies regarding the SWMM and final considerations

Contacts with the different model builders during the study indicate clearly that while the SWMM is continuously improved and additional features are added by Water Resources Engineers (WRE) (in a proprietary form), and the University of Florida (non-proprietary), there is little up-dating activity at present for RRL and UCUR.

For all the reasons indicated above, it is considered that the SWMM has definite advantages compared to the RRL and UCUR models. It should be pointed out that, in some conditions, these latter two models may perform quite well and extensive comparisons made on the test areas as well as the theoretical analyses indicate again that any model can be improved. The advantages of the hydrograph runoff models in general for Canadian practice will be pointed out in Section 3.8 where they will be compared with the Rational Method.

3.5 Selection of Parameters for the SWMM

3.5.1 General

As indicated in Subsection 3.1.2, estimates of parameters used in any hydrologic model can be improved if a large number of samples are available for calibration. In order to demonstrate this, an effort was made to improve the results of the SWMM runoff simulations and several parameters were adjusted by using insight into the physical processes of surface runoff. This was also necessary to compare the SWMM results with those of the Unit Pulse Method where such a procedure was used. Practical computerized optimization techniques are not available at present, but it was felt that the simpler intuitive procedure

TABLE 17. COMPARISON OF COMPUTER PROCESSING TIMES FOR SWMM, RRL AND UCUR MODELS

(IBM 370-168)

Model / Area	SWMM Min. CPU	SWMM No.of Time Steps	SWMM No.of Surface Elements	SWMM No.of Pipes	RRL Min. CPU	RRL No.of Time Steps	RRL No.of Surface Elements	RRL No.of Pipes	UCUR Min. CPU	UCUR No.of Time Steps	UCUR No.of Surface Elements	UCUR No.of Pipes
Oakdale (12.9 acres)	.244	150	27	31	.013	180	NA	14	.839*	87	92	14
Gray Haven (23.3 acres)	.097	72	17	26	0.01	72	NA	11	-	-	-	-
Calvin Park (89 acres)	.243	69	53	58	.021	67	NA	53	.190**	54	225	52

* before programming improvements

** after programming improvements

outlined below would be sufficient to demonstrate the improvement possible when data are available for calibration.

The following general prodecure has been used to adjust the parameters in an optimum manner:

1) Adjustment of Manning's n - This modifies flow retardance and therefore also the peak flow and time to peak. (n for both overland and pipe flow)
2) Adjustment of the volume of total runoff by (i) modifying the infiltration parameters; (ii) modifying the retention storage coefficients.
3) Readjusting Manning's n - This is necessary to compensate for any changes made to the peak runoff during the volume adjustment.

3.5.2 Calibration of SWMM on Calvin Park

The initial storm simulations done using the SWMM for the Calvin Park drainage basin were carried out using the default values given in the table below. These parameters are recommended in the EPA user's manual to describe overland flow resistance, retention storage and infiltration on catchments where measurements of these parameters are not available. The initial simulations using the default values and assuming Mannings's 'n' in the pipes is 0.012, are summarized in Figures 25, 27 and 29, which compare the measured and computed values of the following parameters; peak flow (Qp), time to peak (Tp) and the total runoff volume.

SWMM default parameters compared to calibrated parameters

Parameter	Default Value	Modified Value
Resistance Factor Impervious Area	0.013	0.030
" " Pervious "	0.250	0.375
Retention Storage Impervious " (in)	0.062	0.020
" " Pervious " (in)	0.184	0.092
Maximum Infiltration Rate (in/hr)	3.000	3.000
Minimum " " "	0.520	0.520
Decay Rate of Infiltration	0.00115	0.00115
Impervious Area with no Retention	25%	25%

TABLE 18. RECORDED PEAK FLOWS ON CALVIN PARK AND INITIAL PEAK FLOWS OF THE SWMM COMPARED TO CALIBRATED PEAK FLOWS OF THE SWMM AND QUURM MODELS

		MEAS.	Uncalibrated SWMM		Calibrated SWMM		Calibrated QUURM	
No.	STORM DATE	Qp 1 (cfs)	Qp 2 (cfs)	2/1	Qp 3 (cfs)	3/1	Qp 4 (cfs)	4/1
1	7/25/72-1	38.1	46.0	1.21	40.9	1.08	39.0	1.02
	-2	26.6	32.7	1.22	27.6	1.04	25.1	0.94
3	8/9/72-1	8.2	9.1	1.11	8.9	1.08	9.8	1.20
4	8/9/72-2	18.7	28.6	1.53	24.7	1.32	23.5	1.26
5	8/14/72-1	12.3	15.7	1.28	13.2	1.07	12.0	0.98
	-2	13.2	14.8	1.12	11.5	.87	10.0	0.76
6	8/18/72-1	6.0	4.4	0.73	5.6	.94	6.2	1.03
	-2	3.4	4.7	1.38	4.0	1.18	3.9	1.13
7	8/23/72	19.0	22.0	1.16	19.1	1.00	19.8	1.04
8	6/1/73	9.4	7.0	.75	7.5	.80	9.2	0.98
9	7/12/73	9.0	12.2	1.36	11.5	1.28	12.7	1.41
10	7/26/73	18.7	27.8	1.48	24.4	1.30	24.3	1.30
11	8/1/73	38.6	47.2	1.22	45.6	1.18	42.8	1.11
$\bar{X}$ = MEAN $\frac{\text{Qp calc}}{\text{Qp meas}}$				1.20		1.09		1.09
S = STANDARD DEVIATION of Qp ratios				.22		.16		.16

TABLE 19. RECORDED TIMES TO PEAK FOR CALVIN PARK AND INITIAL TIMES TO PEAK OF THE SWMM COMPARED TO CALIBRATED TIMES TO PEAK FOR THE SWMM AND QUURM MODELS.

No.	STORM DATE	MEAS. Tp 1 (min)	Uncalibrated SWMM Tp 2 (min)	Uncalibrated SWMM 2/1	Calibrated SWMM Tp 3 (min)	Calibrated SWMM 3/1	Calibrated QUURM Tp 4 (min)	Calibrated QUURM 4/1
1	7/25/72-1	13	10	.77	13	1.00	14	1.08
	-2	25	24	.96	25	1.00	26	1.04
3	8/9/72-1	23	21	.91	22	.96	24	1.04
4	8/9/72-2	22	20	.91	22	1.00	23	1.05
5	8/14/72-1	91	87	.96	89	0.98	92	1.01
	-2	181	176	.97	178	.98	182	1.01
6	8/18/72-1	15	12	.80	14	.94	18	1.20
	-2	35	33	.94	35	1.00	39	1.11
7	8/23/72	12	11	.92	12	1.00	13	1.08
8	6/1/73	17	12	.71	14	.82	15	.88
9	7/12/73	16	12	.75	13	.81	15	.94
10	7/26/73	33	24	.73	25	.78	27	.82
11	8/1/73	34	23	.68	27	.79	28	.87
$\bar{X}$ = MEAN $\frac{\text{Tp calc}}{\text{Tp meas}}$				.85		.93		1.0
S = STANDARD DEVIATION of Tp ratios				.11		.09		.11

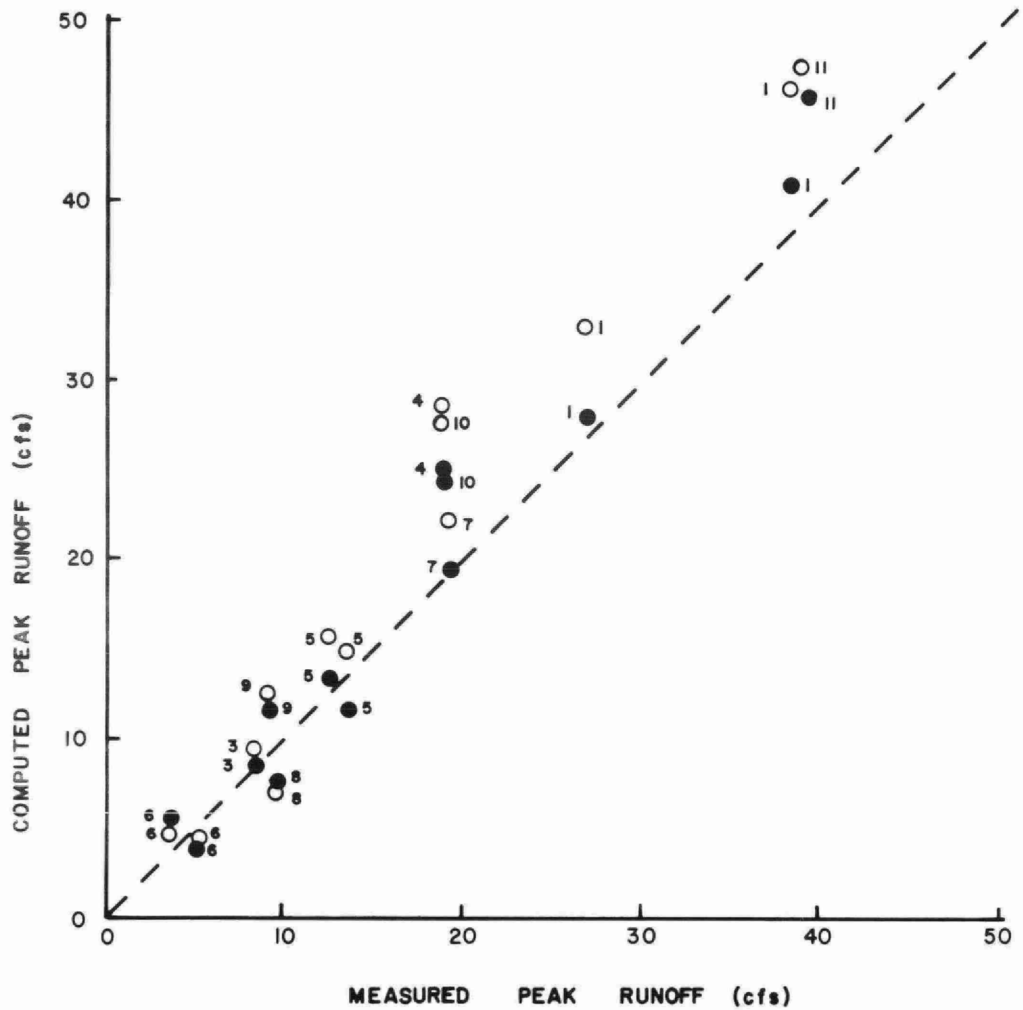
TABLE 20. RECORDED VOLUMES OF RUNOFF FOR CALVIN PARK AND INITIAL VOLUMES OF THE SWMM MODEL COMPARED TO THE IMPROVED VOLUMES OF THE SWMM AND QUURM MODELS.

		MEAS.	Uncalibrated SWMM		Calibrated SWMM		Calibrated QUURM	
No.	STORM DATE	Vol 1 (cu ft)	Vol 2 (cu ft)	$^2/_1$	Vol 3 (cu ft)	$^3/_1$	Vol 4 (cu ft)	$^4/_1$
1	7/25/72	50016	45986	.91	48313	.96	48728	.97
3	8/9/72-1	13002	10221	.78	12593	.97	12752	.98
4	8/9/72-2	22764	19320	.85	22246	.98	21859	.96
5	8/14/72	55104	47205	.85	49580	.90	49943	.91
6	8/18/72	10680	8670	.81	11053	1.03	11278	1.06
7	8/23/72	18210	14608	.81	16762	.92	17306	.95
8	6/1/73	7974	5333	.67	7652	.96	7831	.98
9	7/12/73	7530	8391	1.10	10684	1.42	10928	1.45
10	7/26/73	21264	24593	1.16	26826	1.26	27322	1.28
11	8/1/73	58500	49430	.84	53098	.92	52825	.90
$\bar{X}$ = MEAN $\frac{\text{Vol calc}}{\text{Vol meas}}$				.88		1.03		1.04
S = STANDARD DEVIATION of Vol. ratios				.14		.17		.20

The ten available storms were divided into two similar sets - 5 storms for calibration and 5 storms for validation. Calibration of the model using the above procedure resulted in the modified parameters given in the preceding table. The infiltration parameters remained unchanged since, for the relatively low intensity storms which were used, the pervious area contributed an insignificant amount of runoff. Reduction of the surface storage parameters was necessary to increase the total volume of runoff. This in turn increased the immediate runoff, increasing the peak, which required an increase in the resistance to flow (Manning's n) both for the overland and pipe flow. The model proved more senstitive to modifications in the roughness coefficient for pipe flow, which was increased from 0.012 to 0.017. Furthermore, the time of overland flow, and hence the amount of surface detention storage, was increased by decreasing the equivalent width of the subcatchments by 20%, thereby increasing the overland flow length for the same total area. This adjustment is justified because the so-called "width" of the subcatchments is in most cases not a measurable parameter since subcatchments are generally not regular rectangular shapes as considered by the model.

The modifications described above increased the volume of runoff while at the same time decreasing the peak flow and increasing the time to peak. The results are summarized in Tables 18 to 20 and plotted in Figures 36 to 38. The comparison of computed and recorded hydrographs is presented in Table 21 and Figures 43 and 44.

Comparison of the mean and standard deviations of the ratios of calculated peak flows, time to peak and volumes of runoff and the respective measured values are given in the table below. By reference to the original uncalibrated values of these parameters given in the same table, significant improvement is evident.



Numbers refer to storm identification given in table 19.

Storms used for calibration; 1,3,4,8,10

Storms used for validation; 5,6,7,9,11

○ COMPUTED WITHOUT CALIBRATION USING DEFAULT PARAMETERS.

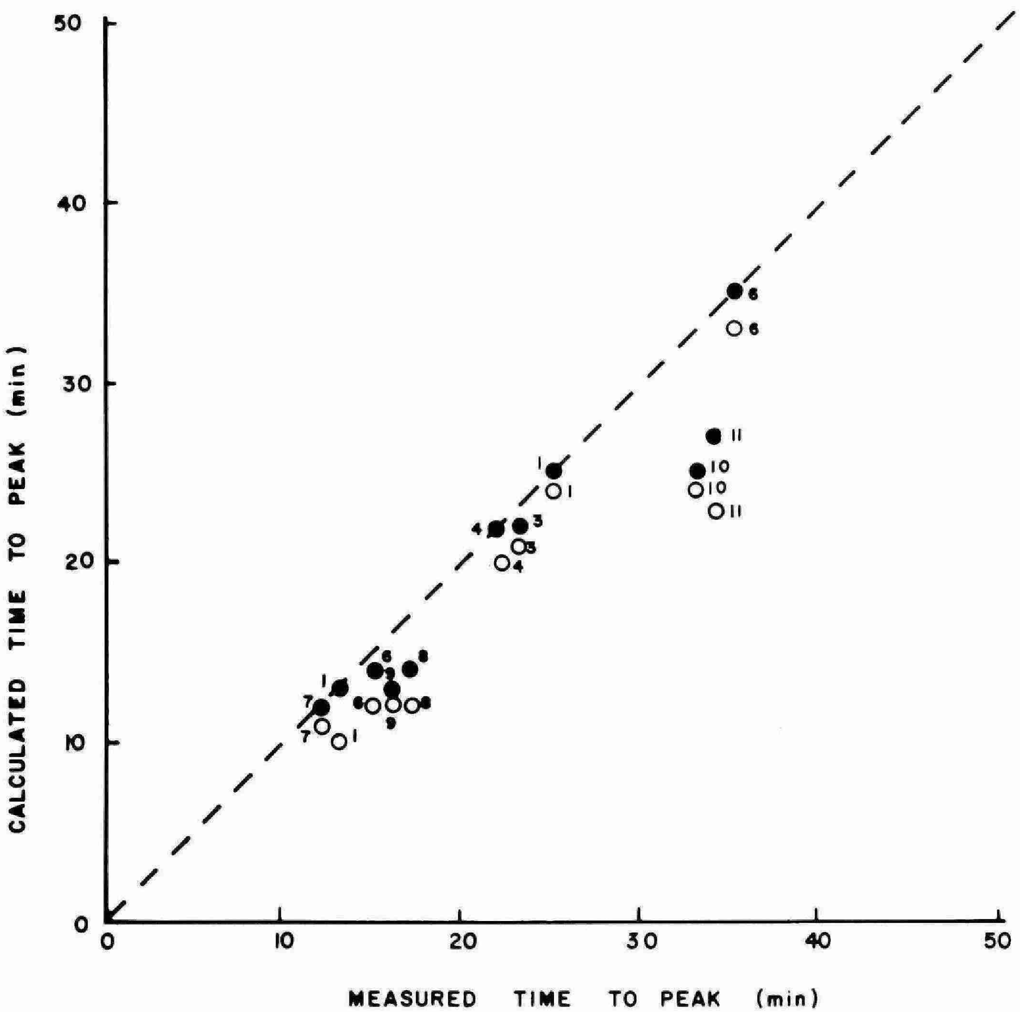
● COMPUTED AFTER CALIBRATION.

CALVIN PARK

SWMM CALIBRATION RESULTS

Qp

FIG. 36



Numbers refer to storm identification given in table 19.

Storms used for calibration; 1,3,4,8,10

Storms used for validation; 5,6,7,9,11

○ COMPUTED WITHOUT CALIBRATION USING DEFAULT PARAMETERS.

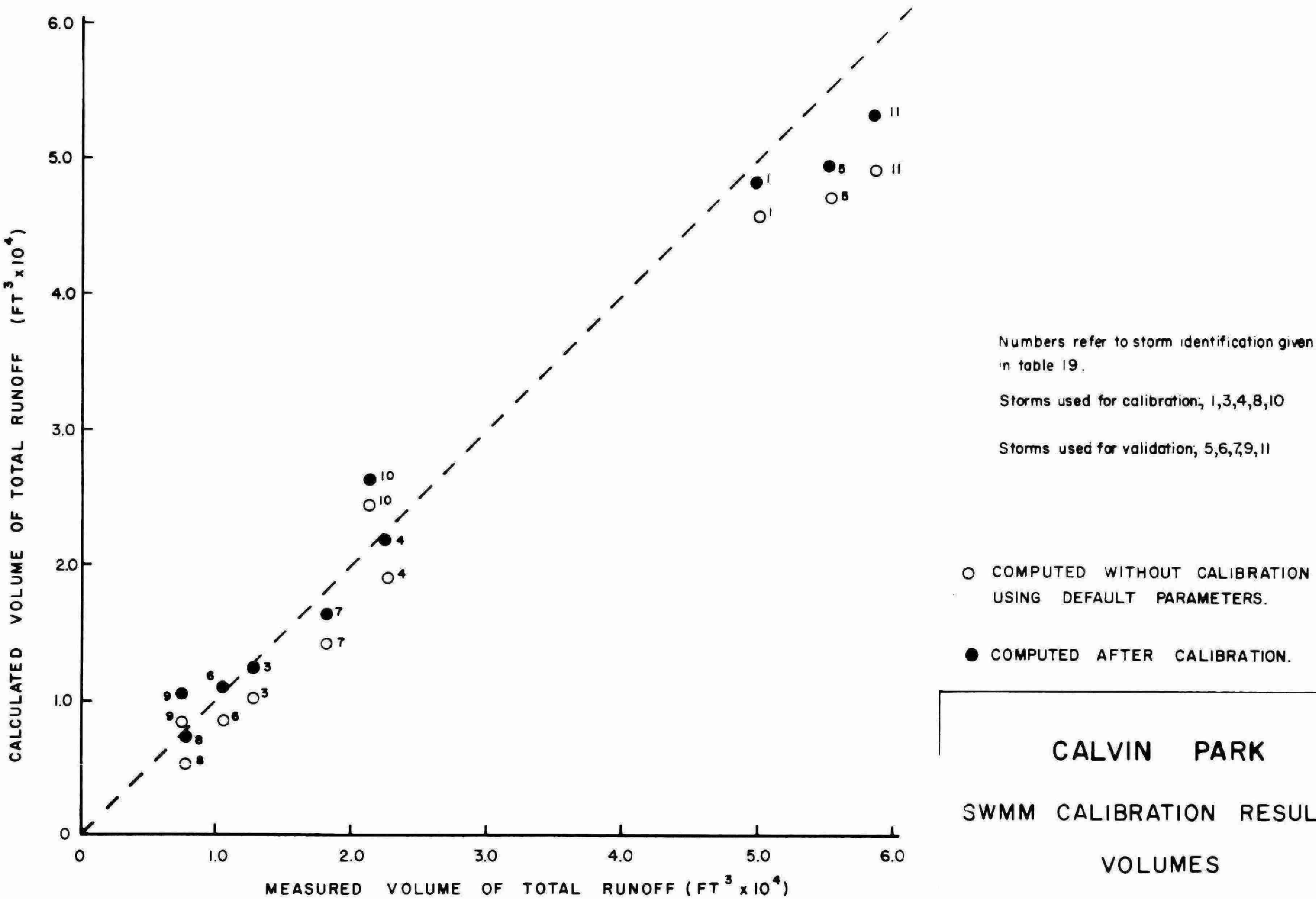
● COMPUTED AFTER CALIBRATION.

CALVIN PARK

SWMM CALIBRATION RESULTS

Tp

FIG. 37

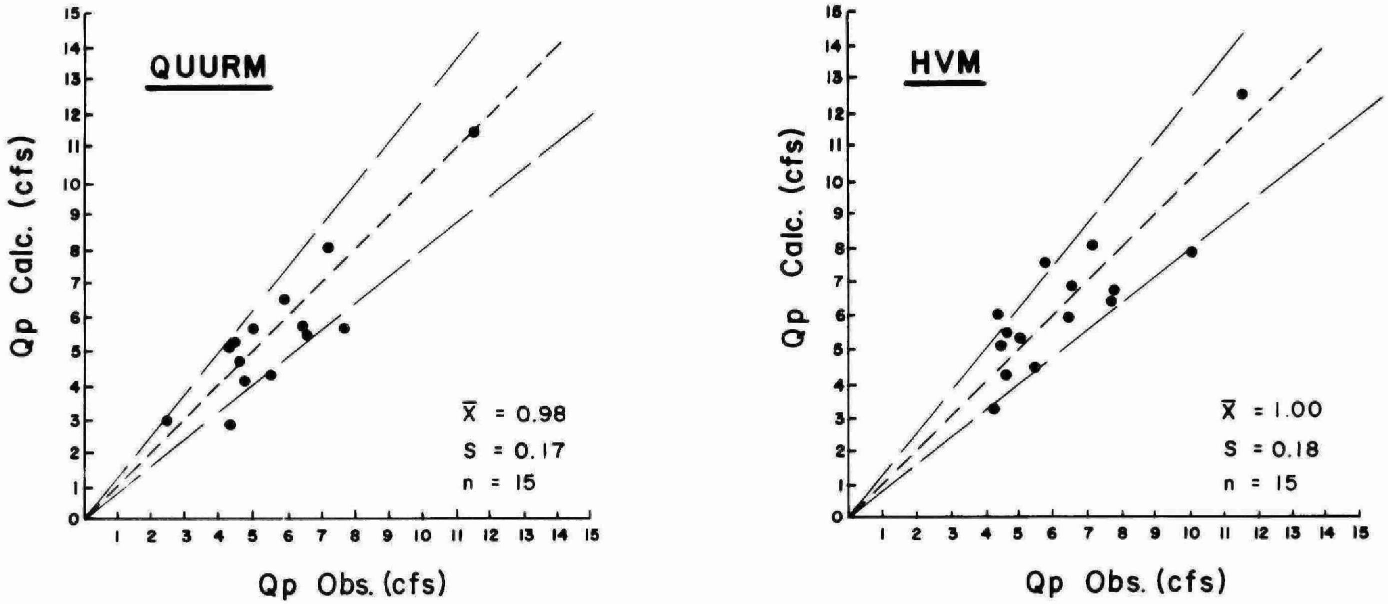


CALVIN PARK

SWMM CALIBRATION RESULTS

VOLUMES

FIG. 38



Qp = peak flow

$\overline{x}$ = mean of $\frac{Qp\ (calc)}{Qp\ (obs)}$

n = no. of observations

Qp(obs) = Qp(calc) — — — —

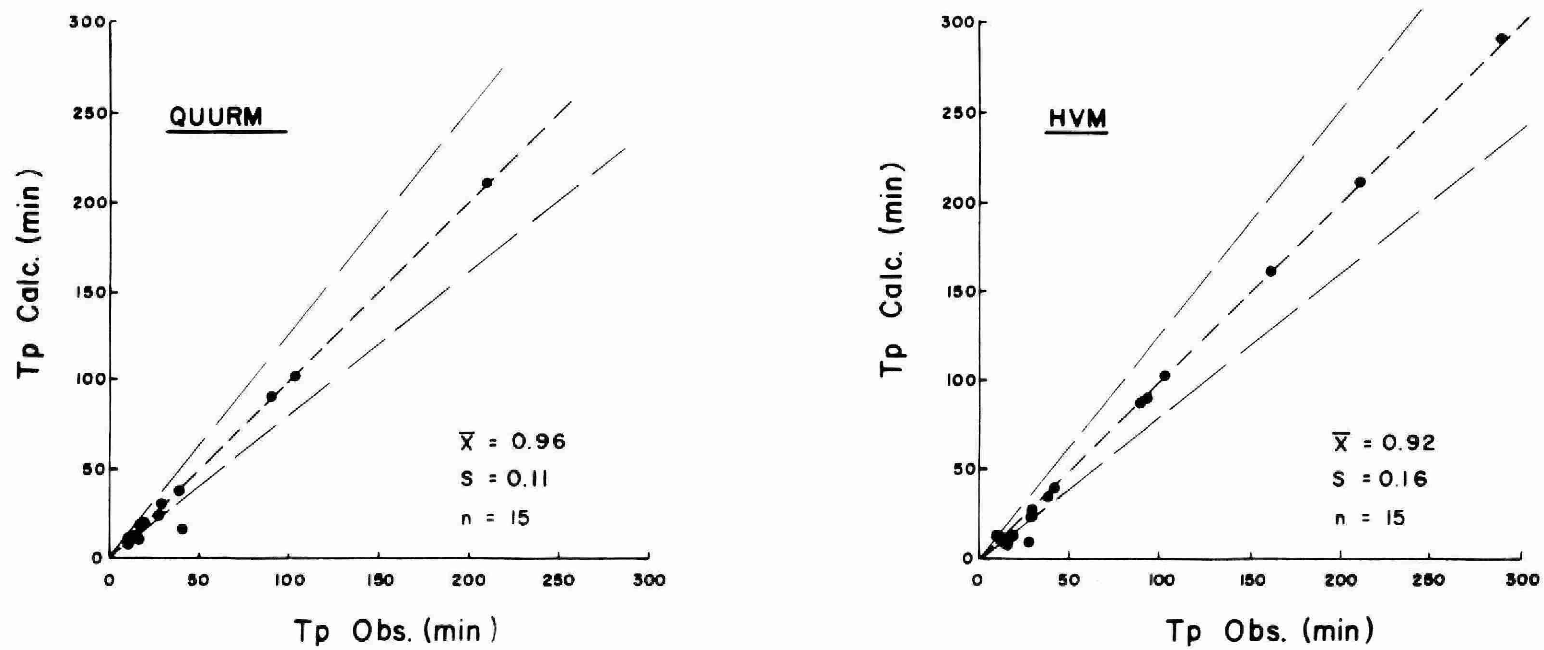
LINES OF ±20% ERROR —— ——

OAKDALE, CHICAGO

Qp CALCULATED vs. Qp OBSERVED

QUURM AND HVM MODELS

FIG. 39



Tp = time to peak

$\overline{x}$ = mean of $\frac{\text{Tp (calc)}}{\text{Tp (obs)}}$

n = no. of observations

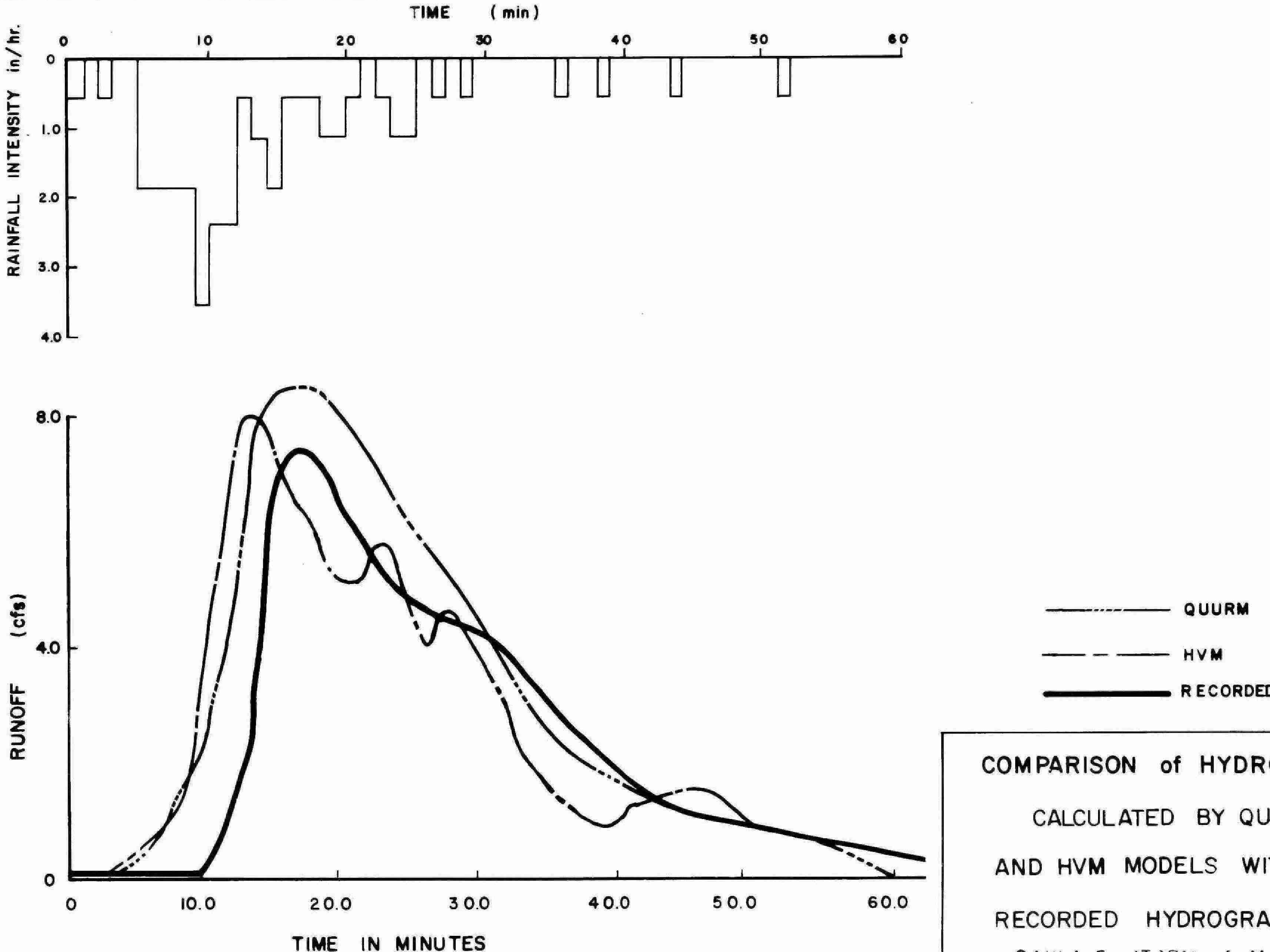
Tp (obs) = Tp (calc) — — — — —

LINES OF ±20% ERROR —— ——

OAKDALE, CHICAGO

Tp CALCULATED vs. Tp OBSERVED QUURM AND HVM MODELS

FIG. 40



COMPARISON of HYDROGRAPHS

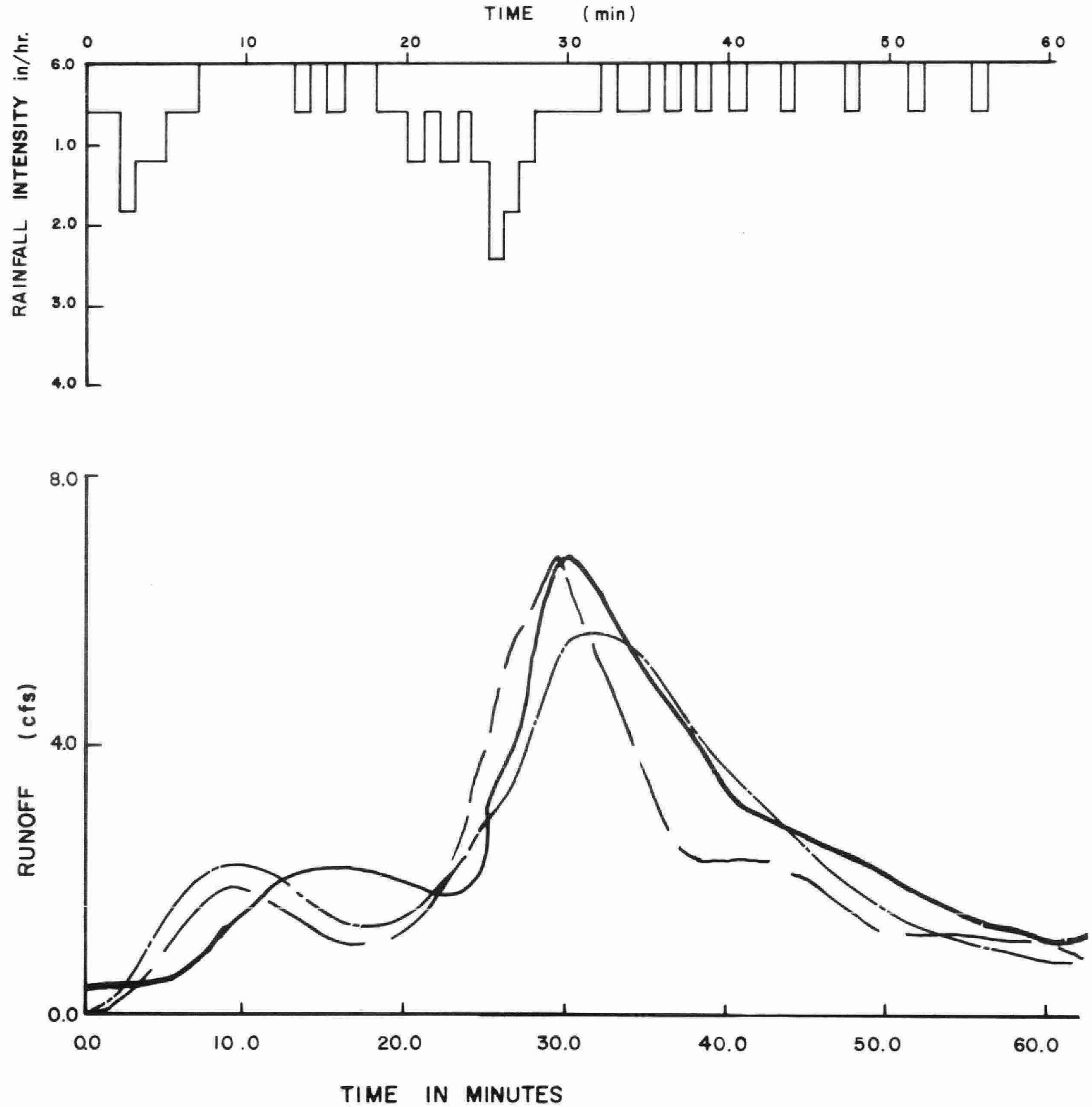
CALCULATED BY QUURM

AND HVM MODELS WITH

RECORDED HYDROGRAPH

OAKDALE STORM of May 19, 1959

FIG. 41



COMPARISON of HYDROGRAPHS CALCULATED BY QUURM AND HVM MODELS WITH RECORDED HYDROGRAPH

OAKDALE STORM of April 29, 1963

FIG. 42

Results of SWMM calibration on Calvin Park	Calibrated SWMM	Uncalibrated SWMM
Mean $\frac{\text{Qp computed}}{\text{Qp measured}}$	1.09	1.20
Standard devistion of QP ratios	.162	.217
Mean $\frac{\text{Tp computed}}{\text{Tp measured}}$	.93	0.85
Standard deviation of Tp ratios	.091	.105
Number of observations	13	13
Mean $\frac{\text{Vol computed}}{\text{Vol measured}}$	1.03	.88
Standard deviation of Vol ratios	.171	.138
Number of observations	10	10

The simple calibration method used here indicates that the SWMM model is quite flexible and improvements on specific basins can be easily made by modifying the physical parameters.

3.5.3 Validation on Oakdale and Gray Haven

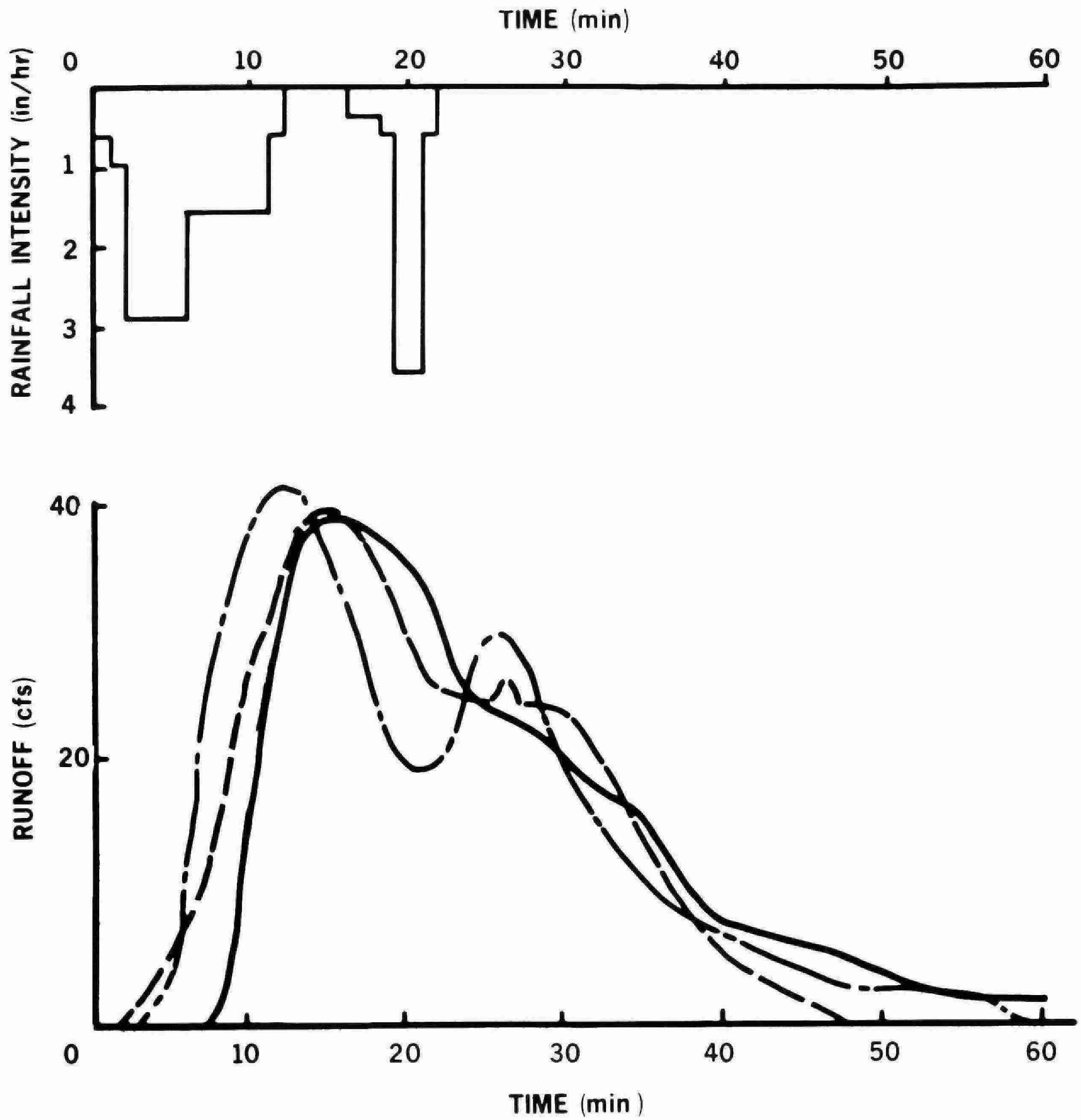
The SWMM with the parameters adjusted in accordance with the results of the Calvin Park calibration was applied to the Oakdale and Gray Haven test areas to determine if improved results would be obtained. A summary of the results of these runs is presented in the tables below:

Results of the calibrated SWMM on Oakdale and Gray Haven

Oakdale	Calibrated SWMM	Uncalibrated SWMM
Mean $\frac{\text{Qp computed}}{\text{Qp measured}}$	1.04	1.08
Standard deviation of Qp ratios	.188	0.200
Mean $\frac{\text{Tp computed}}{\text{Tp measured}}$	.924	.857
Standard deviation of Tp ratios	.135	.176
Number of Observations	17	17

TABLE 21. STATISTICAL COMPARISON OF RECORDED RUNOFF HYDROGRAPHS ON THE CALVIN PARK TEST AREA WITH THE IMPROVED SWMM AND QUURM COMPUTED HYDROGRAPHS

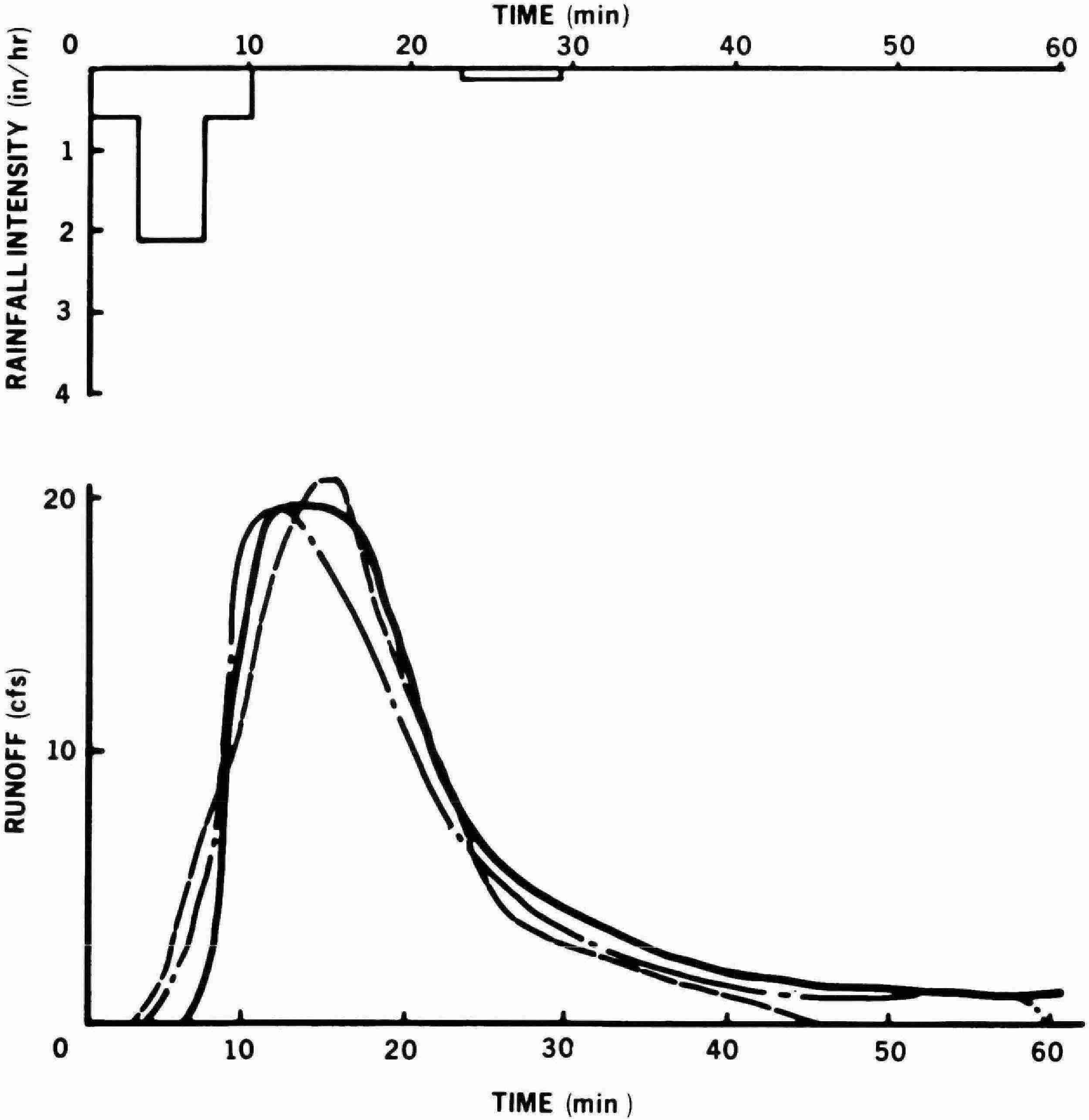
No.	STORM DATE	PARA-METER	Cali-brated SWMM	Uncali-brated SWMM	Cali-brated QUURM
1	7/25/72	R	.876	.826	.958
		RS	.882	.776	.951
		ISE	5.94	8.17	3.80
3	8/9/72-1	R	.939	.878	.911
		RS	.938	.859	.872
		ISE	4.02	6.11	5.82
4	-2	R	.860	.702	.897
		RS	.786	.512	.813
		ISE	8.37	12.65	7.81
5	8/14/72	R	.847	.701	.911
		RS	.882	.727	.930
		ISE	2.90	4.42	2.23
6	8/18/72	R	.941	.789	.967
		RS	.948	.815	.955
		ISE	2.99	5.67	2.79
7	8/23/72	R	.973	.858	.985
		RS	.963	.819	.980
		ISE	3.86	8.64	2.86
8	6/1/73	R	.542	.311	.614
		RS	.546	.335	.508
		ISE	14.18	17.15	14.74
9	7/12/73	R	.674	.448	.765
		RS	.412	.131	.309
		ISE	16.36	19.89	17.71
10	7/26/73	R	.373	.173	.462
		RS	.023	.363	.055
		ISE	19.25	22.75	18.92
11	8/1/73	R	.638	.490	.710
		RS	.463	.193	.571
		ISE	11.31	13.87	10.10



COMPARISON OF HYDROGRAPHS CALCULATED BY QUURM & SWMM CALIBRATED MODELS WITH RECORDED HYDROGRAPH

CALVIN PARK STORM OF JULY 25, 1972

Figure 43



COMPARISON OF HYDROGRAPHS CALCULATED BY SWMM & QUURM CALIBRATED MODELS WITH RECORDED HYDROGRAPH

CALVIN PARK STORM OF AUGUST 23, 1972

Figure 44

Gray Haven	Calibrated SWMM	Uncalibrated SWMM
Mean $\frac{Qp\ computed}{Qp\ measured}$	.98	1.176
Standard deviation of Qp ratios	.235	.219
Mean $\frac{Tp\ computed}{Tp\ measured}$	1.018	.976
Standard deviation of Tp ratios	.055	.095
Number of Observations	14	14

Overall improvement in the results is again evident using the calibrated SWMM although the improvement is not as dramatic as on Calvin Park since the Oakdale and Gray Haven results using the uncalibrated model were already very good. This may be due to the much smaller size of these latter two areas (12.9 and 23.3 acres respectively) compared to Calvin Park (89.4 acres) since the sensitivity of the model to errors in the input parameters would be expected to increase with the length of overland flow and pipe length. The actual size of the area for which calibration becomes important, however, needs to be investigated further. If the overland flow hydrographs are used as input to an accurate trans port routine, the subareas used can be kept small and the resulting errors minimized.

It is apparent that if improvement is desired to reduce the errors indicated in Section 3.1.2, measurements from the watershed under consideration are necessary for the calibration of parameters in each specific case. There are errors inherent in the use of runoff hydrograph models at their present stage of development that cannot be avoided without several years of measurements for the improvement of parameters. (A methodology for this data collection is suggested in Section 3.9.) For design purposes, however, these errors are smaller than those of the Rational Method.

3.6 Proprietary Models

3.6.1 General

As indicated in Table 1 several models for the study of urban runoff are available from private firms. It is felt that although a detailed investigation of these models was not possible, limited information regarding some of them might be of interest. The following models will be briefly considered:

a) The HVM model owned by Dorsch Consult of Toronto (41).
b) The MIT model owned by Resource Analysis Inc. in Cambridge, Mass. (43)
c) The updated SWMM owned by Water Resources Engineers of California (71).
d) The HSP model owned by Hydrocomp of Palo Alto, California (18).

The HVM program is a one-storm event simulation model consisting mainly of an overland flow routine, and a sewer network transport routine. The overland flow routine uses equivalent rectangular surfaces for the watersheds, and accounts for depression storage, according to non-uniform distribution of depth, and infiltration losses, by Horton's formula. A main feature of the model is a sophisticated routing routine for the conduit network which can deal also with surcharged flow and backwater (41).

The updated SWMM owned by WRE features a transport routine which can also deal with flow under surcharge. The hydrographs from the basic subcatchments, however, are derived by a method similar to that of the original program (71). The MIT model uses the kinematic wave method for routing on the equivalent rectangular surfaces and in the collecting system (43).

Details on the capabilities of each model for the simulation of backwater and surcharge conditions, the retention capacity of sewers and the diversion of flow rates at branching structures and the variation of the flow processes with respect to time, can be found in the various publications of these firms and their comparison is beyond the scope of this study.

The HSP system is an advanced version of the Stanford Watershed Model which was the first continuous simulation model. There is a basic difference between all the models presented above or considered in this study and the continuous simulation models. The HSP model maintains a running account of the water in soil moisture storage by adding new rainfall and deducting direct runoff, accretion of ground water and evapotranspiration. The amounts of runoff ground water accretion are expressed as functions of the prevailing soil moisture storage. This is consistent with infiltration theory in which the infiltration capacity is a function of the soil moisture storage. This avoids the arbitrary assumptions for antecedent conditions or an initial infiltration capacity which are common to all single event models. The model needs calibration of more parameters, however, and requires more computer time. Several models for continuous simulation are studied by Canadian hydrologists. Two continuous simulation models have been developed by Professor S. Solomon and Professor N. Kouwen from the University of Waterloo. These models were developed for a study of urbanization on large areas. Other models for rural areas were developed by the University of Laval. Continuous simulation is a more rigorous approach than one event simulation since no arbitrary assumptions are necessary with regard to antecedent moisture conditions. Real storms can be used as an input instead of a design storm and the frequency of flooding can be determined. Because of cost and lack of experience, however, its use in municipal engineering may be considered as a next step after one event hydrograph models are more widely accepted by the practice.

It is not within the scope of this study to do a comparative evaluation of proprietary models. It is felt that municipal engineers who familiarize themselves with the principles of modelling by the use of non-proprietary models will also be better able to assess the merits of proprietary models and continuous simulation. In Appendix IV, the transport routines of the more sophisticated Dorsch and WRE models are compared with those considered in this study.

During this study, it became possible to include comparative runs on the Northwood and Oakdale test areas through the cooperation of Dorsch Consult who provided the results of the HVM model on these areas. It was also possible to compare for Gray Haven the results of the simulation by the MIT model with those of the SWMM and RRL models.

3.6.2 Results with the HVM model

As mentioned above, the Hydrograph Volume Method is a comprehensive urban sewer system simulation model developed by the Dorsch Consult Company of Munich, Germany. The North American rights are owned by Dorsch Consult Limited of Toronto. The company generously agreed to apply their model to the same Oakdale and Northwood test data used for the other models and make the results available to this study.

Although the proprietary character of the model inhibits the disclosure of specific details, the model does include capabilities for simulating backwater, and other conditions as indicated in Section 3.6.1. On the Oakdale and Northwood watersheds, however, the flow processes are determined by the overland flow phenomena, rather than the properties of the small sewer networks. A typical uniform subarea approach, conceptually similar to the UCUR model, is used to subdivide the drainage area and the program output includes the various depths of flow, flow rates, velocities and the height of water in the manholes when surcharging occurs, as well as the hydrographs.

The results of the Oakdale and Northwood test runs are shown in Figures 16, 17, 39, 40, 41 and 42, and Tables 7, 8, 22 and 23. It is evident that for these small catchments, the HVM model gives results equivalent to those of the EPA model. However, the parameters used for the HVM model were apparently not the same on these areas as for the other models, for which default values of the parameters were chosen, as indicated in Section 3.2.3

TABLE 22. RECORDED PEAK FLOWS ON OAKDALE AND PEAK FLOWS COMPUTED BY THE QUURM AND HVM MODELS

No.	STORM DATE	MEAS. Qp 1 (cfs)	QUURM Qp 2 (cfs)	QUURM $^2/_1$	HVM Qp 3 (cfs)	HVM $^3/_1$
1	5/19/59	7.25	8.13	1.12	8.1	1.12
2	7/2/60	4.60	4.61	1.00	4.3	.94
3	7/26/60-1	2.50	2.98	1.19	-	-
	-2	4.30	2.87	.67	3.3	.77
4	9/18/60	5.10	5.68	1.11	5.3	1.04
5	10/14/60	4.50	5.30	1.18	5.1	1.13
6	7/2/62-1	10.10	-	-	7.9	.78
	-2	7.90	-	-	6.7	.85
7	4/17/63-1	4.60	5.02	1.09	-	-
	-2	6.50	5.84	.90	5.9	.91
8	4/19/63	11.60	11.52	.99	12.6	1.09
9	4/29/63	7.80	5.80	.74	6.9	.89
10	4/30/63	6.70	5.77	.86	4.5	.67
11	6/19/63	5.75	4.33	.75	6.5	1.13
12	8/2/63-1	4.85	4.19	.86	5.5	1.13
	-2	5.95	6.58	1.11	7.7	1.29
13	9/22/64	4.45	5.22	1.17	6.0	1.35
$\bar{X}$	= MEAN $\frac{\text{Qp calc}}{\text{Qp meas}}$			.98		1.01
S	= STANDARD DEVIATION of Qp ratios			.17		.17

TABLE 23. RECORDED TIMES TO PEAK ON OAKDALE AND COMPUTED TIMES TO PEAK OF THE QUURM AND HVM MODELS

No.	STORM DATE	MEAS. Tp 1 (mins)	QUURM Tp 2 (mins)	QUURM 2/1	HVM Tp 3 (mins)	HVM 3/1
1	5/19/59	17	18	1.06	14	.82
2	7/2/60	14	12	.86	12	.86
3	7/26/60-1	11	8	.73	-	-
	-2	210	211	1.01	210	1.00
4	9/18/60	27	24	.89	20	.74
5	10/14/60	16	12	.75	8	.50
6	7/2/62-1	92	-	-	91	.99
	-2	162	-	-	162	1.00
7	4/17/63-1	20	22	1.10	-	-
	-2	103	103	1.00	103	1.00
8	4/19/63	42	43	1.02	40	1.00
9	4/29/63	30	31	1.03	29	.97
10	4/30/63	288	292	1.01	293	1.01
11	6/19/63	11	10	.91	14	1.27
12	8/2/63-1	18	19	1.05	14	.78
	-2	90	92	1.02	88	.98
13	9/22/64	38	38	1.00	35	.92
$\bar{X}$	= MEAN $\frac{\text{Tp calc}}{\text{Tp meas}}$			.96		.92
S	= STANDARD DEVIATION of Tp ratios			.11		.16

3.6.3 Results of the MIT model

The MIT model was calibrated by the model builders on the basis of the Gray Haven data. Only results for a few storms are available from the published information. Tables 9 and 10 compare the results obtained by the SWMM and MIT model for the few storms that are available, and the results of the two models are comparable for the specific conditions of the watershed and the storms considered

3.7 Assessment of the Rational Method

3.7.1 General considerations

The Rational Method is the simplest of all models used in the design of urban drainage systems. It can be expressed in the well known formulation

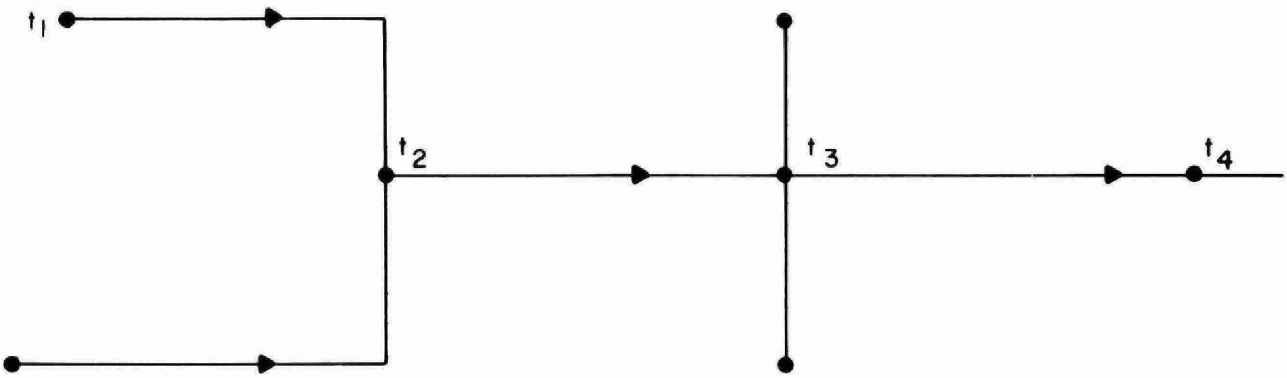
$$Q = CiA$$

in which Q = the peak rate of runoff; C = a constant coefficient related to land use and surface characteristics; i = the average rainfall intensity for a given duration and return period and A = the total contributing area.

The application of the Rational Method formula to storm sewer design is illustrated in Figure 45 and is described in ASCE Manual No. 37 (20).

In comparing the Rational Formula with the runoff hydrograph methods, it is possible to present the following comments:

1) The Rational Formula does not take into account the time distribution of the rainfall.
2) The parameter 'C' lumps all characteristics of the system, such as the degree of imperviousness which is exclusively a subcatchment parameter, with infiltration losses, depression and detention storage, which are also a function of the rainfall characteristics. The selection of 'C' is somewhat arbitrary while the basic parameters of urban runoff models are more objective.
3) For the hydraulic routing corresponding to a given storm event, the Rational Method subsitutes consideration of different rainfalls of decreasing intensities. It does



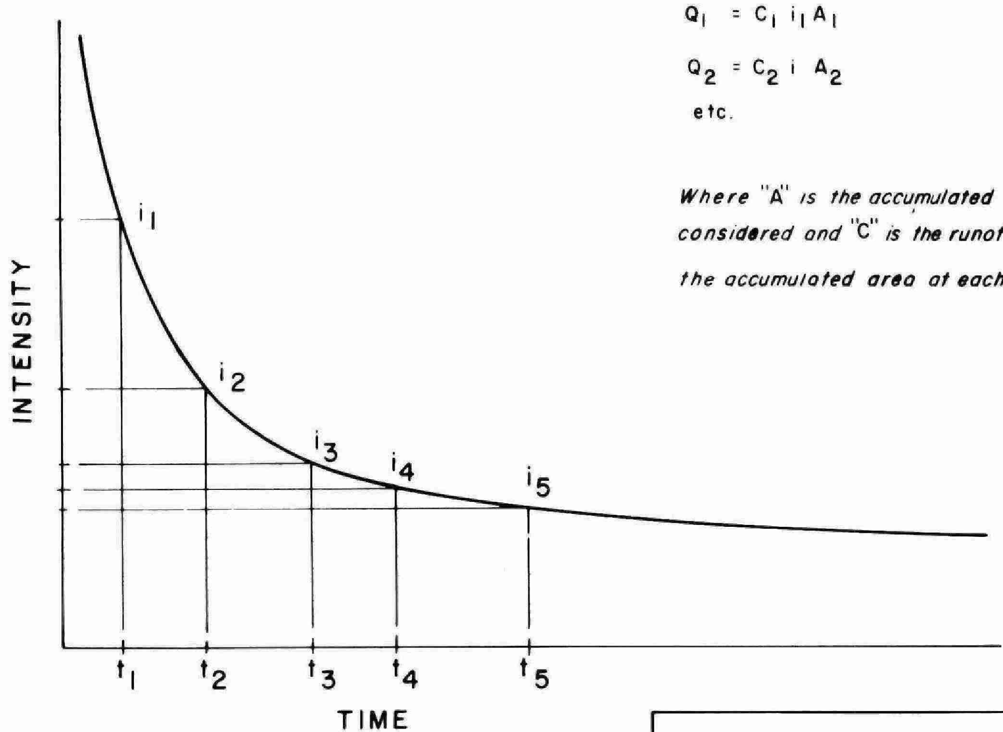
TYPICAL SEWER SYSTEM SHOWING TIMES OF CONCENTRATION FOR VARIOUS LOCATIONS

$Q_1 = C_1 \, i_1 \, A_1$

$Q_2 = C_2 \, i \, A_2$

etc.

Where "A" is the accumulated area to the point considered and "C" is the runoff coefficient for the accumulated area at each point.



TYPICAL INTENSITY-DURATION CURVE SHOWING DERIVATION OF AVERAGE INTENSITIES FOR EACH TIME OF CONCENTRATION

RATIONAL METHOD APPLIED TO STORM SEWER DESIGN

FIG. 45

not give the designer the flow history of a runoff event. It gives only peak values and not hydrographs.

4) The application of the hydrograph methods requires more engineering input than the Rational Method, and also, of course, access to a computer. The Rational Method can be used by junior engineers or technologists with a minimum of training and, although the method has been computerized, it is most frequently applied by hand computation. The advantage of simplicity of this method is offset by the frequency of its misuse and the wide variation of possible results (see Section 2.2)

5) The block rainfall data input into the Rational Method can be obtained from intensity-frequency-duration curves, which are readily available. The hydrograph methods, however, require the development of a design storm.

Points 2 and 3 above are particularly important with regard to the limitations of the Rational Method when methods of storm water management are being considered (Section 2.1). The study of alternatives considering storage, changes of drainage, changes of roof connections, grading, etc. is not possible by the Rational Method.

Attempts have been made to "improve" the Rational Method (61, 62) but apparently none of these methods is being widely used in Canada at present, and they will be mentioned only briefly in the conclusions of this chapter.

3.7.2 Methodology for the assessment of the rational method

The assessment of the Rational Method as a simulation model for peak flows is not possible by the methodology used in this study for hydrograph models because flows resulting from uniform rainfall are fictitious and cannot be checked against measurements. The validity of a method for design purposes cannot be determined unless it is checked against measurements and therefore several methods of assessment have been considered:

1) Comparison of flows calculated for rainfalls of different frequencies with the recorded flows of corresponding

frequencies. This method was suggested by Shaake (65) who, having found that the runoff coefficient 'C' varies with the return period, also suggested empirical relationships for 'C' and a characteristic rainfall averaging time. (See Figure 46.) The method is not applicable where only a limited period of observation is available, however, and it was not used in this study.

2) Calculation of Qp for a maximum average intensity for each measured rainfall during a period equal to the time of concentration (See Figure 46.) This method was applied for all the measured storms using a 'C' coefficient based on the weighted average method recommended in the ASCE Manual No. 9:

$$C_{avg.} = \frac{C\ perv \times A\ perv + C\ imp \times A\ imp}{A\ perv + A\ imp}$$

where C perv is the C value for pervious areas

C perv = 0.1

C imp. is the C value for impervious areas

C imp. = 0.9

It was found that values of C calculated by this method are close to the average C given by Shaake's formula (See Figure 46a) and eliminate the somewhat arbitrary judgment for C based on land use. (See table below.) The difference in these values and those based on land use recommended by the same Manual does not affect the conclusions of this discussion.

Area	Imperviousness %	C Weighted Average	C Shaake's formula
Calvin Park	30.4	.33	.34
Oakdale	45.8	.42	.42
Gray Haven	52.0	.46	.48
Northwood	68.0	.58	.58

From the point of view of the practising engineer, it is also very important to know whether the peak runoff given by the Rational

FROM REGRESSION ANALYSIS ON BALTIMORE AREA DATA:

$$t_l = \frac{1.05 \; L^{0.24}}{S^{0.16} \; IMP^{0.26}} = t_c$$

AND

$$C = 0.14 + 0.65 \, IMP + 0.05 \, S$$

WHERE: t_l = PEAK LAG TIME (mins.)

L = MAXIMUM DRAINAGE LENGTH

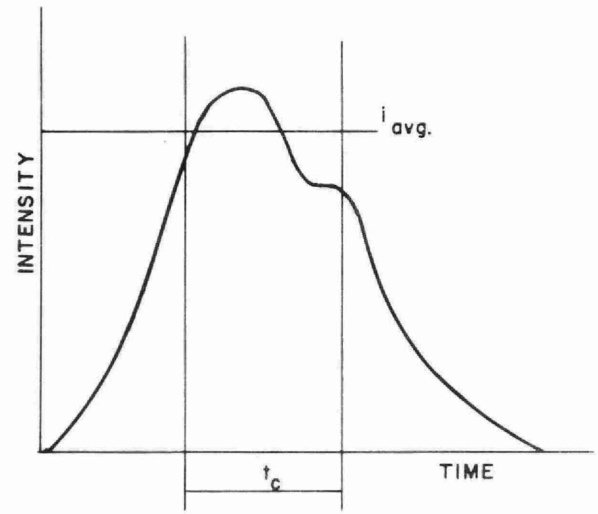
S = SLOPE IN PERCENT

IMP = PERCENT IMPERVIOUSNESS

EMPIRICAL METHOD FOR DERIVING RATIONAL METHOD PARAMETERS t_c AND C

(After SCHAAKE [65])

Fig. 46a



ACTUAL HYETOGRAPH

1. t_c DETERMINED AT OUTLET
2. MAX. AVG. INTENSITY FOUND FOR t_c FROM RECORDED STORM HYETOGRAPH.
3. C, A FROM PHYSICAL DATA
4. Q MAX. = C i A

COMPUTATION OF PEAK FLOW FROM RECORDED RAINFALL USING RATIONAL METHOD

(82)

Fig. 46b

Formula is conservative. This is difficult to determine unless many peak values are measured, but consistency compared to the hydrograph methods is of great interest.

3.7.3 Comparison of Rational Method peak flows with measurements

The results for the different areas are presented in Table 24 and Figures 47 and 48.

Mean ratios of Qp calculated, Qp predicted and standard deviations of these ratios are given below, which can be compared with similar tables for the hydrograph models. (See Section 3.3)

Results of the Rational Method applied to all test areas

Area	No. of Observations	mean $\frac{\text{Qp computed}}{\text{Qp measured}}$	Standard deviation of Qp ratios
Calvin Park	10	1.68	0.261
Oakdale	17	1.37	0.475
Gray Haven	10	1.01	0.233
Northwood	14	0.98	0.349

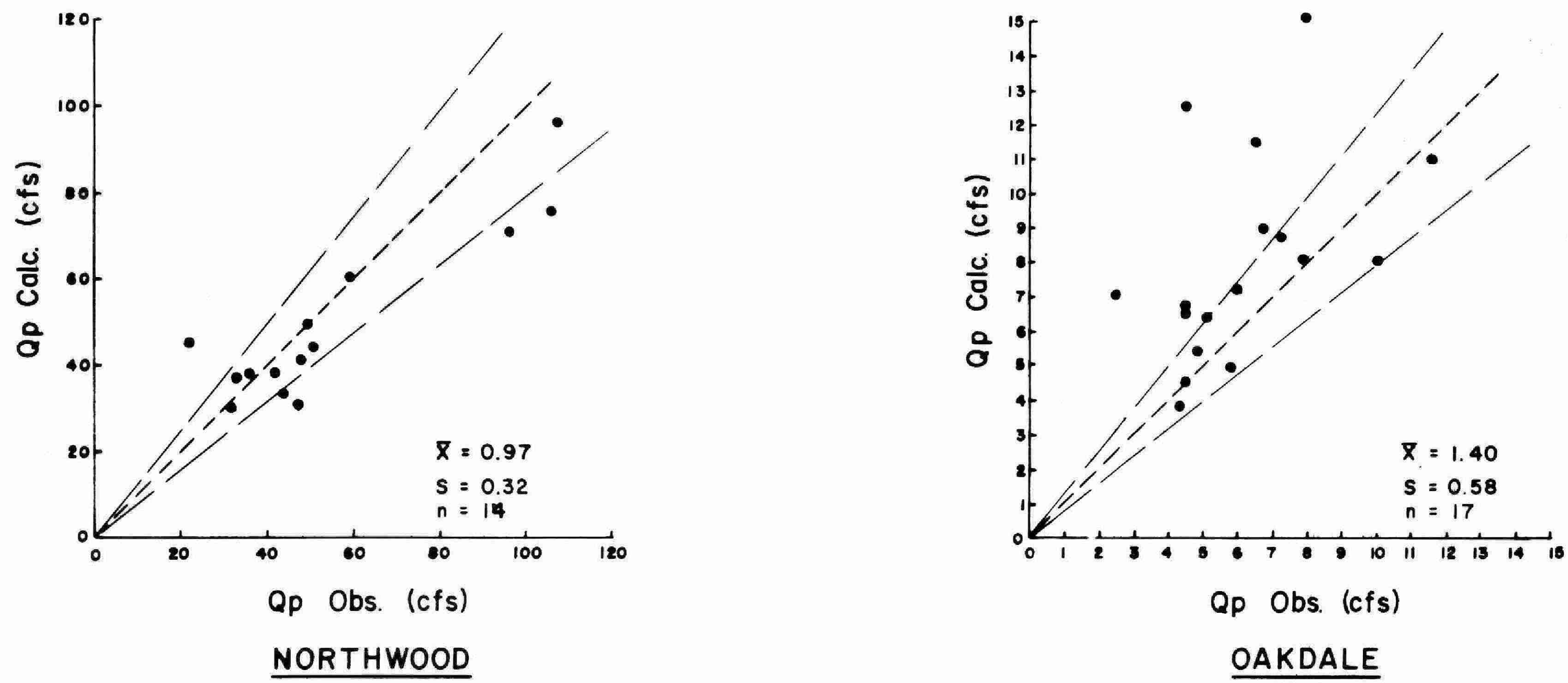
These results show that on some of the areas the method over estimates while on others it under estimates. It is recalled that for the SWMM the mean values of the peak ratios were consistently slightly higher than 1.0. The scatter indicated by the standard deviation is also much larger for the Rational Method.

Measurements of four small watersheds in Philadelphia with areas from 30 to 288 acres were compared by the Philadelphia Water Department against the Rational Method (46). The method of comparison was not described. In this case, the peak gauged flows were larger than those calculated by the Rational Method.

In order to check the Rational Method for larger areas than those already considered, two areas were selected from the publication of Watkins (82). One area is a 611 acre residential development with 121 acres of paved surfaces, for which the average C was computed as 0.350, while the other is a 5,270 acre, largely rural catchment with only 100 acres of pavement for which the average C was computed to be 0.213. Using the same method of comparison as before for 25 storms on

TABLE 24. RESULTS OF USING SWMM PARAMETERS FROM CALVIN PARK CALIBRATION ON COMPUTED OAKDALE PEAK FLOWS

No.	STORM DATE	MEAS. Qp 1	Calibrated SWMM Qp 2	2/1	Uncalibrated SWMM Qp 3	3/1
1	5/19/59	7.25	8.69	1.09	9.11	1.26
2	7/2/60	4.60	4.86	1.06	4.69	1.02
3	7/26/60-1	2.50	3.07	1.23	3.18	1.27
	-2	4.30	3.19	.74	3.14	.73
4	9/18/60	5.10	6.16	1.21	6.17	1.21
5	10/14/60	4.50	5.63	1.25	5.61	1.25
6	7/2/62-1	10.10	8.27	.82	8.21	.81
	-2	7.90	6.40	.81	6.79	.86
7	4/17/63-1	4.60	5.28	1.15	6.18	1.34
	-2	6.50	6.82	1.05	6.52	1.00
8	4/19/63	11.60	10.37	.89	12.69	1.09
9	4/29/63	6.70	5.83	.88	6.87	1.03
10	4/30/63	5.75	5.03	.89	4.80	.84
11	6/19/63	7.80	6.30	.81	6.84	.88
12	8/2/63-1	4.85	5.94	1.22	5.75	1.19
	-2	5.95	7.97	1.34	7.58	1.27
13	9/22/64	4.45	5.60	1.26	6.09	1.37
$\bar{X}$ = MEAN $\frac{\text{Qp comp}}{\text{Qp meas}}$				1.04		1.08
S = STANDARD DEVIATION of Qp ratios				.19		0.20



Qp = peak flow

$\bar{x}$ = mean of $\frac{Qp(calc)}{Qp(obs)}$

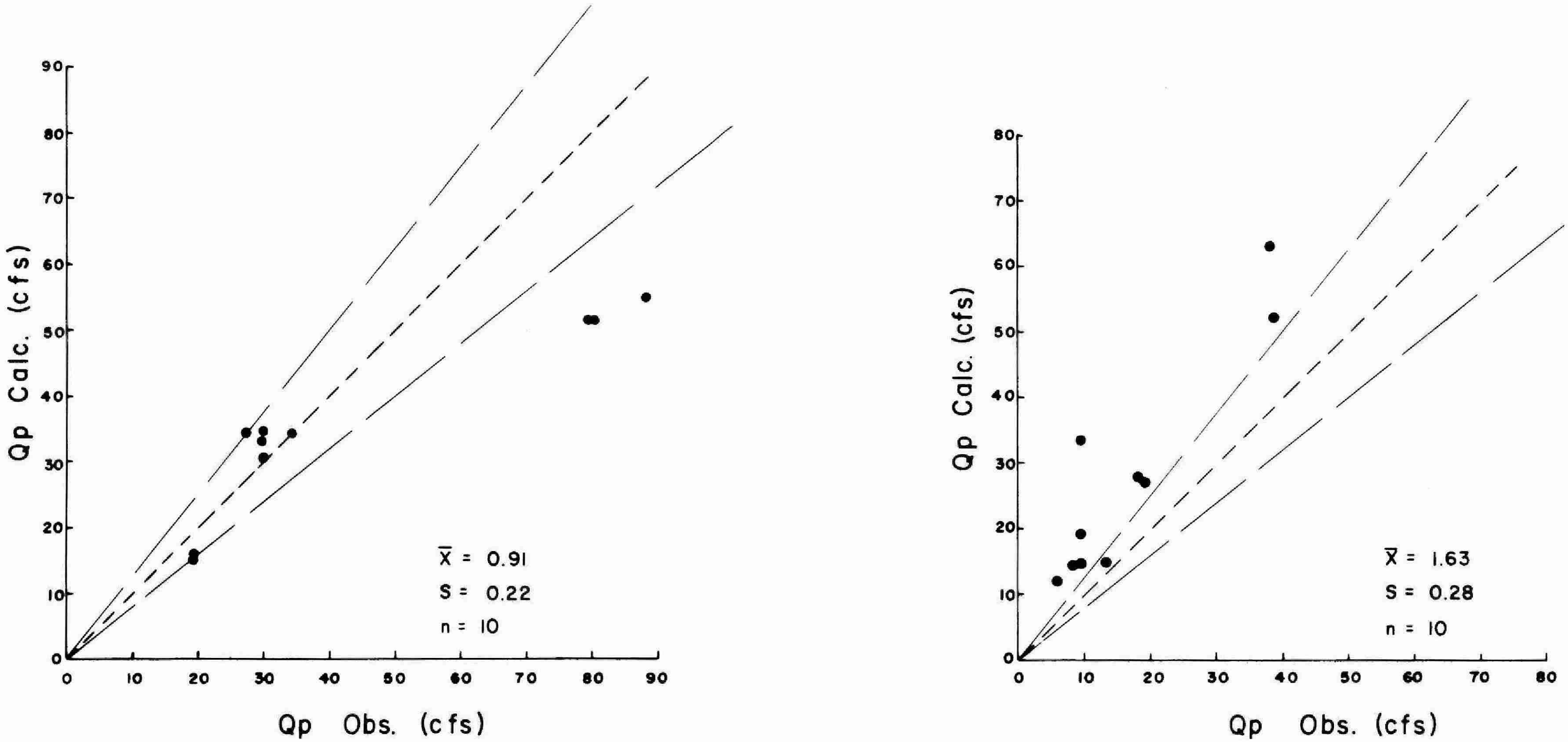
n = no. of observations

Qp (obs) = Qp (calc) – – – – –

LINES OF ± 20% ERROR —— ——

Qp CALCULATED vs Qp OBSERVED ON NORTHWOOD AND OAKDALE USING THE RATIONAL METHOD

FIG. 47



GRAY HAVEN

CALVIN PARK

Q p = peak flow

$\bar{x}$ = mean of $\frac{Qp\,(calc)}{Qp\,(obs)}$

n = no. of observations

Qp (obs) = Qp (calc) — — — — —

LINES OF ±20% ERROR —— ——

Qp CALCULATED vs Qp OBSERVED ON GRAY HAVEN AND CALVIN PARK USING THE RATIONAL METHOD

FIG. 48

the smaller area and 12 storms on the larger, the mean peak ratios and standard deviations are given below.

Results of Rational Method applied to two large Catchments

Area	No. of Obser.	Runoff Coeff. C	mean $\frac{Qp \text{ computed}}{Qp \text{ measured}}$	Standard deviation of Qp ratios
611 acres	25	.350	1.96	.418
5270 acres	12	.213	2.89	2.867

For both catchments, the calculated values are considerably larger than the measured ones. While this might be the result of an incorrect C value, the scatter of the results indicated by the large standard deviations is independent of the choice of C.

These results indicate that the Rational Method is inconsistent and not appropriate for the simulation of runoff from a real storm event.

The inconsistencies in using the Rational Method as a model can be explained if it is considered that the coefficient "C" includes the effects of two unrelated factors which should be expressed separately; namely, storage and losses. A simple formula defining these aspects separately, advanced only for illustration, would be (11)

$$Q = C_o A (I - L)$$

where L represents the rainfall losses associated with the intensity I and antecedent conditions and C_o expresses the attenuation of the flood peak due to storage.

For cases when comparatively small rainfall intensities are used, such as those measured on the test areas in this study, losses on pervious areas are significant compared to the average rainfall intensity I. For the design conditions, however, these losses are relatively small, because of the high intensities and wet antecedent conditions which result in significant runoff from pervious areas. The inability of the Rational Method to account for this variation is evident, leading to the poor results obtained above for this method when used as a model of recorded events.

3.7.4 The Rational Method as a design tool

The question remains, however, as to the appropriateness of the

TABLE 25. RESULTS OF USING SWMM PARAMETERS FROM CALVIN PARK CALIBRATION ON COMPUTED OAKDALE TIMES TO PEAK

No.	STORM DATE	MEAS. Tp 1	Calibrated SWMM Tp 2	$^2/_1$	Uncalibrated SWMM Tp 3	$^3/_1$
1	5/19/59	17	16	.94	15	.88
2	7/2/60	14	10	.72	7	.50
3	7/26/60-1	11	10	.91	6	.55
	-2	210	209	.99	209	.99
4	9/18/60	27	22	.81	22	.82
5	10/14/60	16	11	.69	9	.56
6	7/2/62-1	92	95	1.04	92	1.00
	-2	162	163	1.00	162	1.00
7	4/17/63-1	20	14	.70	14	.70
	-2	103	102	.99	101	.98
8	4/19/63	42	39	.93	40	1.05
9	4/29/63	30	30	1.00	29	.97
10	4/30/63	288	292	1.01	292	1.01
11	6/19/63	11	9	.82	8	.73
12	8/2/63-1	18	16	1.19	16	.89
	-2	90	90	1.00	89	.99
13	9/22/64	38	37	.97	36	.95
$\bar{X}$	= MEAN $\frac{\text{Tp comp}}{\text{Tp meas}}$			.92		.86
S	= STANDARD DEVIATION of Tp ratios			.14		.18

TABLE 26. RESULTS OF USING SWMM PARAMETERS FROM CALVIN PARK CALIBRATION ON COMPUTED GRAY HAVEN PEAK FLOWS

No.	STORM DATE	MEAS. Qp 1	Calibrated SWMM Qp 2	2/1	Uncalibrated SWMM Qp 3	3/1
1	6/5/63	80.7	65.3	.81	78.0	.97
2	6/10/63	79.0	65.3	.83	78.2	.99
3	6/14/63-1	30.8	31.0	1.01	31.4	1.02
	-2	23.2	31.2	1.34	33.5	1.44
4	6/20/63-1	29.6	33.0	1.11	34.1	1.15
	-2	22.6	31.4	1.38	33.7	1.49
5	6/29/63	27.2	29.7	1.09	30.9	1.13
6	8/1/63-1	88.1	56.8	.64	63.3	.72
7	8/14/63-1	34.7	34.9	1.01	38.5	1.11
	-2	16.8	19.8	1.17	21.6	1.29
	-3	17.4	16.9	.99	19.1	1.12
8	8/18/64	19.6	13.1	.67	16.1	.82
9	8/2/65	30.3	30.6	1.02	34.6	1.15
10	8/12/66	19.3	12.3	.64	15.1	.78
$\bar{X}$	= MEAN $\frac{Qp\ comp}{Qp\ meas}$			.98		1.18
S	= STANDARD DEVIATION of Qp ratios			.24		.22

TABLE 27. RESULTS OF USING SWMM PARAMETERS FROM CALVIN PARK CALIBRATION ON COMPUTED GRAY HAVEN TIMES TO PEAK

No.	STORM DATE	MEAS. Tp 1	Calibrated SWMM Tp 2	2/1	Uncalibrated SWMM Tp 3	3/1
1	6/5/63	41	39	.95	39	.95
2	6/10/63	41	39	.95	39	.95
3	6/14/63-1	22	22	1.00	21	.96
	-2	44	44	1.00	43	.98
4	6/20/63-1	22	23	1.04	21	.96
	-2	44	44	1.00	43	.98
5	6/29/63	28	30	1.07	29	1.04
6	8/1/63-1	12	14	1.16	13	1.08
7	8/14/63-1	16	16	1.00	15	.94
	-2	48	48	1.00	47	.98
	-3	110	110	1.00	109	.99
8	8/18/64	14	14	1.00	12	.86
9	8/2/65	35	35	1.00	34	.97
10	8/12/66-2	24	26	1.08	25	1.04
$\bar{X}$	= MEAN $\frac{\text{Tp comp}}{\text{Tp meas}}$			1.02		.98
S	= STANDARD DEVIATION of Tp ratios			.06		.05

Rational Method as a design method. As discussed in Subsection 3.1.3, the design flow is the result of simulating the runoff due to a hypothetical design storm with a selected return frequency. In Section 3.3, however, it can be seen that, in general, most of the rainfalls used in the assessment of the models have a much smaller average intensity than these design storms. The design storm for the Rational Method is a rainfall with a uniform intensity and a duration equal to the time of concentration. Design storms for hydrograph methods have a wide variation in intensity and have been developed by the method indicated in Figure 4. In order to compare the results of a hydrograph model using a design storm hyetograph and the Rational Method, a design storm with the storm characteristics of Chicago and a magnitude determined for an arbitrary location in Southern Ontario was used as input to the SWMM model on three test areas. The outlet flows were compared with those given by the Rational Method using the C values indicated previously and the appropriate block rain for comparison, and the results are given below. Of course, the resulting peak flows calculated by the two methods are not directly comparable, since the design storms are quite different.

Comparison of design flows calculated by the SWMM and the Rational Method

Area	Acres	Runoff Coeff. C	Peak flow using Rational Method	Peak flow from SWMM and design storm hyetograph
Calvin Park	89.4	.33	65.3 cfs	79.7 cfs
Oakdale	12.9	.42	17.3 "	15.9 "
Gray Haven	23.3	.46	34.1 "	25.4 "

The hydrograph method indicates a large design flow for the 89.4 acre Calvin Park test area while the Rational Method indicates a large flow for the Oakdale and Gray Haven catchments which are 12.9 and 23.3 acres respectively. This difference probably results from the consideration of saturated antecedent moisture conditions with the design storm method due to the long rising limb of low intensity rainfall combined with the high peak intensities in the design storm hyetograph (see Figure 4). Because of this condition, a high net intensity of rainfall will occur on the pervious areas after the infiltration rate (in the

TABLE 28. RECORDED PEAK FLOWS ON ALL TEST AREAS COMPARED TO PEAK FLOWS FOR THE RATIONAL METHOD.

OAKDALE

No.	STORM DATE	MEAS. Qp 1 (cfs)	Rational Method Qp 2 (cfs)	2/1
1	5/19/59	7.3	8.7	1.20
2	7/2/60	4.60	12.5	2.72
3	7/26/60-1	2.5	7.1	2.83
	-2	4.3	3.8	.89
4	9/18/60	5.1	6.4	1.26
5	10/14/60	4.5	6.8	1.51
6	7/2/62-1	10.1	8.2	.81
	-2	7.9	15.2	1.92
7	4/17/63-1	4.6	6.5	1.42
	-2	6.5	11.4	1.75
8	4/19/63	11.6	10.9	.94
9	4/29/63	7.8	8.2	1.05
10	4/30/63	6.7	9.0	1.34
11	6/19/63	5.8	4.9	.85
12	8/2/63-1	4.9	5.4	1.10
	-2	6.0	7.3	1.22
13	9/22/64	4.5	4.6	1.02
$\bar{X}$ = MEAN $\frac{Qp\ calc}{Qp\ meas}$				1.40
S = STANDARD DEVIATION of Qp ratios				.58

NORTHWOOD

No.	STORM DATE	MEAS. Qp 3 (cfs)	Rational Method Qp 4 (cfs)	4/3
1	3/26/64	36.0	38.5	1.07
2	11/25/64	42.2	38.5	.91
3	12/26/64	48.8	41.2	.84
4	7/5/65	107.1	96.2	.90
5	7/15/65	47.4	31.6	.67
6	7/28/65	22.3	45.4	2.04
7	8/1/65-1	51.2	44.0	.86
8	-2	59.3	60.5	1.02
9	8/4/65-1	96.2	71.4	.74
10	-2	106.2	75.6	.71
11	8/8/65	33.2	37.1	1.12
12	8/19/65	32.7	30.2	.92
13	9/1/65	44.6	33.0	.74
14	9/24/65	49.8	49.5	1.00
$\bar{X}$ = MEAN $\frac{Qp\ calc}{Qp\ meas}$				.97
S = STANDARD DEVIATION of Qp ratios				.32

TABLE 28. (CONT'D)

GRAY HAVEN			Rational Method		CALVIN PARK			Rational Method	
No.	STORM DATE	MEAS. Qp 5 (cfs)	Qp 6 (cfs)	6/5	No.	STORM DATE	MEAS. Qp 7 (cfs)	Qp 8 (cfs)	8/7
1	6/5/63	80.7	51.9	.64	1	7/25/72	38.1	63.6	1.67
2	6/10/63	79.0	51.9	.66	3	8/9/72-1	8.2	13.9	1.70
3	6/14/63	30.8	31.4	1.02	4	8/9/72-2	18.7	28.0	1.50
4	6/20/63	29.6	33.5	1.13	5	8/14/72	13.2	15.6	1.18
5	6/29/63	27.2	34.6	1.27	6	8/18/72	6.0	12.0	2.00
6	8/1/63-1	88.1	55.2	.63	7	8/23/72	19.0	27.4	1.44
7	8/14/63	34.7	34.7	1.00	8	6/1/73	9.4	14.8	1.57
8	8/18/64	19.6	16.1	.82	9	7/12/73	9.0	19.2	2.13
9	8/2/65	30.3	34.7	1.15	10	7/26/73	18.7	33.6	1.80
10	8/12/66-2	19.3	15.1	.78	11	8/1/73	38.6	52.2	1.35
$\bar{X}$ = MEAN $\frac{\text{Qp calc}}{\text{Qp meas}}$				0.91					1.63
S = STANDARD DEVIATION of Qp ratios				0.22					0.28

order of 0.5 in/hr to 1.0 in/hr) has been deducted. There will, therefore, be a significant volume of rainfall on pervious areas available to depression storage and overland flow during the design storm peak intensities. This runoff from pervious areas is delayed by the characteristically high depression storage capacity and flow resistance factor on pervious areas. However, as the size of a drainage area increases, so does the time lag between the peak rainfall and the peak runoff at the downstream outlet. This delay in the peak flow allows some of the downstream pervious areas to contribute to the peak flow at the outfall, under design conditions.

The size of area required to have this pervious area contribution to the peak flow depends upon the rainfall intensity, surface slopes and infiltration capacity of the specific case considered and this phenomenon was not investigated further in this study.

In the above comparison, the Rational Method cannot account for this change in the degree of pervious contribution from the small Oakdale catchment to the larger Calvin Park catchment while the SWMM can, which explains why the Rational Method does not give consistent results relative to the other method. This also points out once again that the Rational Method does not necessarily give conservative design flows.

3.7.5 General considerations

It can be concluded from the foregoing, that a simple judgment on whether or not the Rational Method gives safe results is not possible. Experienced engineers have always obtained satisfactory results by empirical methods. The above results and the analysis of the enquiries described in Section 2 show, however, that if the method is used without considerable experience, it may lead to errors.

The cost of data collection for the Rational Method is very close to that of hydrograph methods. The cost of engineering including computer time for the latter is higher, however, and varies with the depth of the analysis and sophistication of the transport model used.

Although during the previous discussions models were considered mainly from the point of view of the accuracy of prediction, there are

also other aspects to be considered when comparing the Rational Method to runoff hydrograph methods.

Input parameters are more objective in the runoff hydrograph methods and satisfactory and consistent results can be obtained provided the user has a background in basic hydrology. While the Rational Method gives only peak flows, runoff hydrograph methods give complete hydrographs at any desired point in the system and allows the investigation of different storm water management techniques such as storage and ponding, and alternative design considerations can be analyzed cheaply and quickly. A runoff hydrograph model is also more flexible, being applicable to the analysis of existing systems for which real storm events can be simulated and problem areas accurately located, as well as to intial design.

By using a more complex model, it is necessary to obtain a better understanding of the phenomena involved. The 'C' value in the Rational Method avoids this consideration of the real phenomena and the method leads only towards the choice of a pipe size. If one adopts a hydrograph method, such factors as slope, degree of imperviousness, soil characteristics, velocity in the drainage system, local ponding and so on have to be considered, and <u>the attention is directed towards the cause of the runoff characteristics rather than the gross effects</u>. This role of hydrograph models is by no means a minor factor in the choice of a design methodology.

3.8 Conclusions on Runoff Models

1. The results of evaluating the models using a large number of storm events clearly indicate that the possible errors resulting from the use of the models are larger than indicated in the available literature. The possible errors incurred by the use of the Rational Method, however, are even larger. It is considered, therefore, that the models are sufficiently well developed at present to give satisfactory results for practical applications. This is especially true for the implementation of new management techniques for which the Rational Method is totally unsuitable.

2. The Road Research Laboratory model (RRL) has the advantage that it is simpler than the other models. However, it also has the serious drawback of not considering the runoff contribution of pervious areas which can become significant when the percentage of imperviousness is low, the area large or the rainfall intensities high as with a design storm, for example.
3. The University of Cincinnati Urban Runoff Model (UCUR) has the more accurate surface routing routine for impervious surfaces of the three models considered, but a logical error in the determination of depression storage supply on pervious areas, and, especially, the use of a simple time offset sewer routing technique are serious drawbacks for applications to large areas.
4. The Storm Water Management Model (SWMM) gives the best overall performance of the models considered. Although the manner of calculating the surface flow rate tends to over estimate the detention storage, thus retarding the runoff, this can be corrected by the judicious choice of compensating parameters such as pipe roughness. The SWMM has the advantages of a more sophisticated pipe transport routine, compatability with an existing quality model, and the continued updating and improvement becoming available for this model through the continued research and practical applications in the United States.
5. The QUURM gives results as good as the SWMM and has the advantage of requiring the calibration of only one parameter. This model is also very suitable to the study of the effects of urbanization on surface runoff. Although there is little practical experience with the QUURM at present and the documentation has not yet been completed, this model can be applied effectively to storm sewer design, especially when coupled with an accurate transport routine.

6. The use of the Rational Method for minor applications is acceptable when new management techniques such as ponding are not considered, since the errors for small areas cannot be significant in terms of the required pipe size. However, it cannot be assumed, out of hand, that the Rational Method always over estimates the peak runoff since the results of this study show that the method is inconsistent in this respect.

3.9 Measurement on Urban Watersheds

Although it is considered that, at the present state-of-the-art, existing urban runoff models have definite advantages compared with the Rational Method, it is understood that the modelling techniques are continuously improving. Once a user becomes familiar with one model he can improve it by his own experience or may be more receptive to similar new techniques. New models, however, have to be tested by comparison with measurements and it is advisable to create a data bank of experimental data for the specific conditions (soils, land use, precipitation) of each city.

At present, there are few urban experimental watersheds, and it is likely that many Canadian municipalities will soon wish to begin measurements to check and calibrate models for their own use. Philadelphia, for example, already has a program for continuous monitoring at 29 gauging stations and rainfall data collection at 30 locations. The drainage areas studied vary in size from 30 acres to 5,360 acres (75).

For this study, a limited number of measurements were done during August - November, 1973 on an experimental watershed in the Brucewood subdivision of North York. The period of time was too short to obtain a record of significant events and measurements will continue in 1974. An effort was made to develop instrumentation and monitoring procedures which, in the long run, may reduce the cost of such data collection. A brief description of the equipment and the test area is given below.

3.9.1 Brucewood test area

The area selected is the Brucewood subdivision located in the

Borough of North York in Metropolitan Toronto. This 38 acre subdivison was constructed in the late 1960's and it is typical of suburban single-family developments of that period. The 39 inch storm sewer outfall drains into the east Don River.

The monitoring station is located in a stilling well behind a 4 foot sharp crested weir constructed across the foot of the apron of the outlet structure. The water level above the crest of the weir is measured by a Manning "Dipper" Transmitter. This instrument senses the water level by means of a small electric probe which, upon contact with the water surface, closes the circuit to a D.C. motor that begins retracting the probe. As soon as the probe begins to retract, the circuit is broken and the probe falls by its own weight to the water surface and the cycle is repeated. The duration of this cycle is approximately 4 seconds. Water levels are converted into a D.C. pulse of between 1.66 and 13.33 seconds duration corresponding to 0 and 30 inches of water above the weir crest, respectively. This pulse is then transmitted over leased telephone lines to a recorder conveniently located in the offices of James F. MacLaren Limited, 5 miles away.

Rainfall is measured by a tipping bucket rain gauge. The pulse from each tip of the bucket is also transmitted over telephone lines to a recorder in the same control panel as the level recorder. The first impulse from the rain gauge starts both the rainfall and water level recorders, each running at a chart speed of 6 in/hr. The charts continue to run for 2 hours after the last pulse from the rain gauge after which time both recorders shut down.

This installation is similar in principle to those in Northwood and Oakdale (Section 3.3). The advantages of this type of instrumentation are that rainfall and runoff are always accurately synchronized, the remote recorders reduce the time and labour involved in travelling to the site to change charts, and the high chart speed gives the 1 minute resolution or less required on small watersheds.

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A P P E N D I X   I

Canadian Design Practice

Questionnaire

(Please complete and return this form to)
James F. MacLaren Ltd.
435 McNicoll Ave.,
Willowdale, Ontario.
M2H 2R8

(a survey by James F. MacLaren from an Urban Runoff Study for Canada Centre for Inland Waters)

A. GENERAL INFORMATION

1. Public Agency

Name of Municipality ______________________

Province, County ______________________

Agency or Department ______________________

Name of Director (Chief Engineer, Head of Department)

____

Future Correspondence should be directed to:

____

Address: ______________________

____

2. Location and Development

Population ______________________

Total Area ______________________

Developed Area ______________________

Percentage High Residential ______________________

Percentage Medium Residential ______________________

Percentage Low Residential ______________________

Percentage Commercial and Industrial ______________________

Receiving Water Bodies

Lakes ______________________

Creeks ______________________

Rivers ______________________

3. Design Activities and Facilities

(Check the appropriate space)

Main Runoff Design Work with ____ Old combined sewers
____ Recent combination sewers
____ Recent storm sewers
____ Drainage planning
Other (specify) ____________________

____

Record on older System ____ Very good
____ Good
____ Incomplete

Method of Keeping New Records____ Plans
____ Data Bank
Other (specify) ____________________

____

Runoff Computations ____ Done mainly through Consultants
____ Done mainly through Developers
____ Done mainly through In-house Group (specify) ________________

____

____ Combined approach (specify)

____

	Yes	No
Do your use a computer in your design work?	___	___

Computer Facilities Available____ In-House
____ Through Terminals
____ At Consultants
____ At Local Universities
____ None

Comments ____

____

____

B. EXISTING STORM DRAINAGE SYSTEM

4. Ponding in Residential Areas

Mark Appropriate space using:

√ Actual Observed

X Considered Acceptable

		Water Ponds in Back Yards	Water Ponds at Inlets	Water Enters Basements	Water Just Covers Street	Water Rises Above Curbs	Manhole Cover Pops Off
a) Frequency							
Times	4						
Per	3						
Year	2						
	1						
Years	2						
Between	3						
Occurrences	5						
	10						
	15						
No Occurences Observed							
b) Period of the Year							
Spring							
Summer							
Fall							
Winter							

Comments ______________________________

5. Maintenance Problems With Residential Storm Sewer Systems

(Check Appropriate Space)

Problem Was: -	Biggest	Normal	Infrequent
General Cleaning of grates, inlets and/or catch basins.			
Collapse of old storm sewers, rebuilding catch basins and outlet trouble			
Children or Animals			
Flooding Intersections and basements			
Clean Storm Sewers			
Freezing			
Roots			

Comments ______________________________

____

____

____

6. Materials Used for Storm Sewer Construction
(Approximate Percentage)

Concrete Pipe ____________

Steel Pipe (Corrugated) ________

Asbestos Cement Pipe ________

Others (Specify) __________

Comments on present trends ______________________________

____

____

____

7. Inlets and Gutters

	Yes	No
Are the inlets depressed	___	___

If both depressed and non-depressed specify approximate percentage of depressed inlets ______________

Average inlet depression in inches ____________

Average inlet length in feet ____________

Maximum width, gutter flow in feet ____________

Average spacing of inlets ____________

Comments ____

8. Methods of Snow Disposal

_____ Dumping into receiving water bodies

Lake ________________

River ________________

Other (Specify) ______________________________

Comments ____

9. Problems Associated with Freezing and Cold Weather ______________

Estimated Cost to Municipality per year ______________________

Comments ____

C. GENERAL DESIGN POLICY

10. Engineering Criteria for Design of Stormwater Drainage Facilities:

	Yes	No
(a) Has your jurisdiction adopted criteria?	___	___
(b) Do the criteria require that the runoff rate from an area (where drainage improvements are planned) be no greater than that of the area in its natural (undeveloped) state?	___	___
(c) Is runoff rate specified to not exceed a particular fraction of the rainfall?	___	___
(d) Are copies of your Stormwater Drainage Criteria available?	___	___
(e) Are criteria being developed now (for your jurisdiction)?	___	___
(f) Do the criteria (or will they) require on-site detention?	___	___
(g) Is it (or will it be) mandatory by law or administrative directive that the criteria be satisfied in order to secure permits for building and/or sewer construction?	___	___

11. Design Standards

Design according mainly to the ASCE Manual ____________

Design according to Specific Standards ____________

Other (specify) ____

Safety Standards ______________________________________

Comments on Standards ____

____

____

____

12. Methods for Runoff Computations

____ Rational Method

____ Other Specify

____ Computations by Hand

____ Computations by Hand and Computer

Comments ____

____

____

____

13. Design Storm Frequency

Residential Areas ____________________________

Commercial and Industrial Areas ________________

Culverts for Driveways ________________________

Minor Roads ________________________

Major Roads (i.e. Arterials) __________

Freeways (if any) ____________________

Comments ____

____

____

____

14. Master Drainage Design (If Completed Recently or Underway) (Check appropriate Space)

	Yes	No
Were major and minor systems clearly defined?	___	___
Are low frequency flows (Less than 1/50) used?	___	___
Are alternatives with open channel and closed conduits compared?	___	___

Selection on economical basis ______________________________

Other Criteria (specify) ______________________________

Degree of surcharge or surface ponding permissible on minor system:

__________ several times a year

__________ once in one year

__________ once in several years

14. (continued)

	Yes	No
Has the sewer system been observed under design storm conditions?	___	___
Does it provide the protection desired under design condition?	___	___
When design calls for intermediate pipe size, is occasional flooding considered (i.e., choosing the smaller pipe with its lower initial cost instead of using the next largest size)	___	___

Cost estimate for providing storm sewers for residential areas.

____

____

Comments: ______________________________________

____

____

____

15. Detention

	Yes	No
Was detention used for reduction of flow peaks?	___	___
Do you feel detention is required?	___	___

If detention facilities have been constructed in your jurisdiction, who owns the land and facility?

Agency__________

Private_________

Other (Specify)______________________________________

Give an estimate of the number of facilities now being used

Roof top________

Parking lot_____

Park ___________

Schoolground____

Other___________

16. Storm (combined) Sewer Use Regulations
(Check Appropriate Space)

Roof drainage is discharged	Present Policy	Policy Prevailing last 25 years
a) Onto property for seepage into soil	______	______
b) Into street gutters	______	______
c) Into storm sewers or surface ditches	______	______
d) Other (specify)	______	______
Foundation drains are connected to		
a) Sanitary sewers	______	______
b) Storm sewers	______	______
c) Combined sewers	______	______
d) Other (specify)	______	______

Comments on Regulations ______________________________

____

____

____

Maximum catch basin spacing ______________

Minimum sewer size ______________

Minimum sewer cover ______________

Maximum distance from end of drainage area to first inlet __________

Method of Inlet Design:

ASCE Manual No. 9 _______

Other (Specify) ______________________________

____

Sewer alignment distance from centreline of road ______________

Maximum distance between manholes ______________

	Yes	No
Are catch basins proposed in new development?	___	___
Are double house connections allowed or not?	___	___
Are losses at bends or in manholes considered in sewer design or compensated for?	___	___

Comments ______________________________

____

____

____

17. Application of Rational Method

Inlet Time   Maximum (specify) ____________________

    Minimum (specify) ____________________

(Specify)

Land Use	Coefficient of Runoff
________	____________
________	____________
________	____________
________	____________
________	____________
________	____________

Rational Formula Used:

a) Only for Peak Flow ______________

b) For Triangular Hydrograph__________

	Yes	No
Is Inlet Design Method used?	___	___

Design Velocities	Earth Channel	Lined Channel	Pipe
Maximum			
Minimum			

Comments ____

____

____

____

18. Characteristic Storm Data - Intensity Duration Curve

	5	10	30	60	Min.
2 year					
5 year					
10 year					

Comments ____

____

____

____

D. ATTITUDES, ENVIRONMENTAL ASPECTS

19. Environmental Aspects Considered

Change in peak flow downstream __________

Change in ground water regime __________

Change in sediment load __________

Impact on wildlife __________

Pollution of receiving water body __________

Recommended measures (specify) __________

Comments __________

20. Problems of Concern and Research Needs (check appropriate space)

	No Need	Interested	High Priority
New Runoff Models	____	____	____
Optimization Techniques	____	____	____
Improved Land-Use	____	____	____
Pollution for Urban Runoff	____	____	____
Detention and Storage	____	____	____
Design Storms	____	____	____
Urban Erosion	____	____	____

Comments __________

21. Other Problems

Major changes in design policy in the last years.

Comments __________

APPENDIX II

Tabulation of Questionnaire Results

APPENDIX 2

TABULATION OF QUESTIONNAIRE RESULTS

The replies received from those cities with populations greater than 100,000 precede those with less than 100,000 population in each section. The sections are listed in numerical order and the content of each section is given below in the Index.

INDEX TO TABLES

(Section numbers are printed on upper right hand corner of tables).

Section No.	Topic
1	Public Agency
2	Location and Development
3	Design Activities and Facilities
4	Ponding in Residential Areas
5	Maintenance Problems with Residential Storm Sewer Systems
6	Materials Used for Storm Sewer Construction
7	Inlets and Gutters
8	Methods of Snow Dispoasl
9	Problems Associated with Freezing
10	Engineering Criteria for Design of Stormwater Drainage Facilities
11	Design Standards
12	Methods for Runoff Computations
13	Design Storm Frequencies
14	Master Drainage Design
15	Detention
16	Storm Sewer Use Regulations
17	Application of Rational Method
18	Characteristic Storm Data
19	Environmental Aspects Considered
20	Problems of Concern and Research Needs
21	Other Problems

LIST OF PARTICIPATING MUNICIPALITIES

Atlantic Provinces	Bathurst, N.B. Fredericton, N.B. Halifax, N.S. St. John's, Nfld.
Quebec	Anjou Cap-de-la-Madeleine Chicoutimi Hull Laval Montreal Pierrefonds Quebec City St. Laurent Ste.-Foy Salaberry de Valleyfield Sherbrooke
Ontario	Brantford Guelph Hamilton Kingston Kitchener Mississauga North Bay Ottawa St. Catharines Sarnia Scarbororugh Sudbury Timmins Thunder Bay Toronto Waterloo Windsor
Western Provinces	Calgary, Alta. Regina, Sask. Vancouver, B.C. Winnipeg, Man.
Northern Areas	Anchorage, Alaska Fairbanks, Alaska Whitehorse, Yukon

SECTION: 2 Location and Development	Population (x1000)	Total Area (x 100 acres)	Developed Area (x 100 acres)	% of TOTAL DEVELOPED AREA High residential	Medium residential	Low residential	Commercial, industrial and institutional	Receiving Water Bodies Lakes	Rivers	Creeks
HALIFAX	125	157	77	15	35	40	10	Atlantic Ocean		
LAVAL	250	607	180	-	-	-	-	-	Des Prairies Des Mille Iles	
QUEBEC CITY	188	234	88	2.1	3.9	6.4	14.5	-	St. Lawrence	
MONTREAL	1320	395	355	----	*58	----	34	-	St. Lawrence Des Prairies	
OTTAWA	300	289	246	0.7	0.9	15.6	10.2	Deschenes Dows	Rideau Ottawa	Graham, Green's Pinecrest Sawmill
SCARBOROUGH	360	448	269	5	5	55	35	Ontario	Rouge	Highland Massey
TORONTO	690	269	173	3	31	16	50	Ontario	Don Humber	
MISSISSAUGA	175	716	-	-	-	-	-	Ontario	Credit	Etobicoke Cooksville
ST. CATHARINES	110	247	110	---	*75	----	25	Ontario		12 mile Welland Canal
WINDSOR	200	300	150	-	-	-	20	St. Clair	Little River	Turkey
HAMILTON	304	348	200	1	4	30	15	Ontario		Red Hill Chedoke Stoney
KITCHENER	122	164	910	---	*40	----	12	Erie	Grand McIntyre	Schneider
THUNDER BAY	109	100	192	10	-	90	20	Superior	Neebing Kaministiquia	McVicars
WINNIPEG	540	1408	957	---	*22	----	8.8	Winnipeg	Assiniboine, Red Seine, La Salle	several
REGINA	145	200	154	-	-	-	-	Wascana		Wascana
CALGARY	425	1004	402	2	2	18	18	Glenmore Reservoir	Elbow, Bow Western Irrigation Canal	Nose Fish
VANCOUVER REG.	1100	-	-	-	-	-	-	Pacific	Fraser	

*combined percentage

SECTION: 2 Location and Development	Population (x 1000)	Total Area (x 100 acres)	Developed Area (x 100 acres)	Percentage High residential	Percentage Med. residential	Percentage Low residential	Commercial, Industrial and Institutional	Receiving Water Bodies		
								Lakes	Creeks	Rivers
ST. JOHN'S	93	74	49	2.4	3.6	40.2	15.2	Lake and Harbour		
BATHURST	17	282	-	-	-	-	-	Harbour & Chaleur Bay	Several	Several
FREDERICTON	44	320	79	2	2	40	15.7		Several	St. John
ANJOU	36	35	21	---	*51	---	49		Pluvial	St. Jette
CHICOUTIMI	37	75	29	5	80	15	10			Saguenay Chicoutimi Du Moulin
SAINTE-FOY	68	203	66	3.8	2.5	27.9	9.2			Rouge,Lorette St. Laurent
SHERBROOKE	82	52	45	6.3	20.9	25	13.7			St. Francois & Magog
CAP-DE-LA-MADELEINE	34	43	17	40	25	15	20	1	3	St. Maurice St. Laurent
VALLEYFIELD	31	82	82	24.4	26.8	39.0	9.8			St. Laurent
SAINT LAURENT	63	106	68	26	22	19.3	26.8		Yes	
HULL	63	63	36	-- *4.8 --			9.3		Several	Ottawa Gatineau
PIERREFONDS	35	61	61	8.72	15.3	50.6	25.55			Des Prairies
KINGSTON	60	75	60	5	2.7	54.8	37.5	Ontario	Cataraqui	St. Lawrence
NORTH BAY	50	826	58	0	-- *66.6 --		33.3	Nipissing Trout Lake	Several	
SUDBURY	99	736	184	5	20	65	10	Ramsey, Minnow Nepaawin,Robinson	Junction	
TIMMINS	43	7840	60	0.83	9.2	15.3	13.3	Pearl,Porcupine Bobs, Gillies	Town	Mattagami Porcupine
GUELPH	63	173	74	---	*36	---	14		Hanlon Torrance	Eramosa Speed
BRANTFORD	66	116	61	---	*59	---	33			Grand
WATERLOO	39	165	55	-	-	-	40		Laurel & Colonial	Grand
SARNIA	57	117	85	---	*38	---	28.4	Huron	Talfourd	St. Clair

* Combined Percentage Figure

SECTION: 3 Design Activities and Facilities	MAIN RUNOFF DESIGN WORK WITH					RECORD ON OLDER SYSTEM			METHOD OF KEEPING NEW RECORDS			RUNOFF COMPUTATIONS				COMPUTER FACILITIES				
HALIFAX	X		X					X	X		Micro-film	X	X	X	X	No			X	X
LAVAL		X	X					X	X			X				No				
QUEBEC CITY	X	X	X				X		X					X		No				
MONTREAL	X	X		X			X		X	X	Micro-film			X		Yes			X	
OTTAWA			X					X	X					X		No	X	X		
SCARBOROUGH			X	X			X		X			X		X		Yes	X	X		
TORONTO	X					X			X					X		Yes	X			
MISSISSAUGA			X						X				X			No				
ST. CATHARINES				X	Water-course encl.		X		X				X			No	X			X
WINDSOR	X	X	X	X			X		X			X	X	X	X	No				
HAMILTON	X		X	X			X		X				X	X		Yes	X			
KITCHENER			X					X	X			X				No				
THUNDER BAY	X		X	X			X		X				X	X		No		X		X
WINNIPEG	X		X	X				X	X			X				Yes			X	
REGINA			X				X		X				X	X		No				
CALGARY			X				X		X			X		X	X	No	X	X		
VANCOUVER REGION	X	X	X				X		X		Profi-les, Flows			X		No				X

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SECTION: 3

SECTION: 3	MAIN RUNOFF DESIGN WORK WITH					RECORD ON OLD SYSTEM			METHOD OF KEEPING NEW RECORDS			RUNOFF COMPUTATIONS					COMPUTER FACILITIES				
Design Activities and Facilities	Old Combined Sewers	Recent Combination Sewers	Recent Storm Sewers	Drainage Planning	Other	Very Good	Good	Incomplete	Plans	Data Bank	Other	Consultants	Developers	In-House	Combined Approach	Design Work by Computer	In-House	Terminals	Consultants	Universities	None
ST. JOHN'S				X				X	X					X		No			X	X	
BATHURST	X				Open Ditches		X		X			X				No			X		
FREDERICTON			X					X	X						X	No				X	
ANJOU			X			X			X			X				-					
CHICOUTIMI	X		X	X		X			X					X		No					X
SAINTE-FOY	X	X	X					X	X					X		No	X				
SHERBROOKE	X		X	X		X			X					X	50%	No					
CAP-DE-LA-MADELEINE			X				X		X					X		No					
VALLEYFIELD	X	X	X	X			X		X					X		No				X	
SAINT LAURENT	X	X	X	X				X	X					X		No					
HULL	X		X	X				X	X			50%		50%	X	Yes			X		
PIERREFONDS			X			X			X					Montreal		No					X
KINGSTON			X					X	X			Subdiv.		X	X	No			X	X	
NORTH BAY			X					X	X			Subdiv.		X	X	No					
SUDBURY			X				X		X			Subdiv.		X	X	No	X			X	
TIMMINS			X					X	X			Consult. for Dev.		X	X	No					
GUELPH			X					X						X		No				X	
BRANTFORD			X	X			X		X				X			No					
WATERLOO			X	X				X	X					X		No			X	X	
SARNIA			X	X				X	X			Subdiv.		X	X	No					

SECTION: 4 Ponding in Residential Areas	WATER PONDS IN BACK YARDS			WATER PONDS AT INLETS			WATER ENTERS BASEMENTS			WATER JUST COVERS STREET			WATER RISES ABOVE CURBS			MANHOLE COVER POPS OFF		
	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year
HALIFAX	4	-	All	*4	-	All	4	-	All	*4	-	All	4	-	All	4	-	All
LAVAL	-	-	-	-	-	-	1	2	Sp	-	-	-	-	-	-	-	-	-
QUEBEC CITY	-	-	-	-	-	-	*3	-	Sp Su	3	-	-	-	None Obs.	-	-	None Obs.	-
MONTREAL	-	none obs.	-	-	*10	Su	-	*10	Su	-	*10	Su	-	*10	Su	-	*10	Su
OTTAWA	Varies from property to property from no problem up to 10 times per year																	
SCARBOROUGH	1	-	Sp	1	-	Sp	2	-	Su	1	-	Su	-	5	-	-	10	Fa
TORONTO	-	-	Sp	*1	-	Su	-	*5	Sp,Su Fa	*1	-	Sp Su	-	*5	None Obs.	-	-	Su
MISSISSAUGA	*1	-	-	-	None Obs.	-	-	*10	-	-	*10	-	-	None Obs.	-	-	None Obs.	-
ST. CATHARINES	*1	-	Su	*1	-	Su	1	*5	Su	*1	-	Su	-	5,*10	Su	-	5,*10	Su
WINDSOR	4	-	All	4	-	All	4	-	All	4	-	-	-	-	-	-	-	-
HAMILTON	*4	-	Sp	-	*15/5	Su	4	-	Sp Su	*1	-	Su	*1	-	Su	-	*15	Su
KITCHENER	2	7	Sp Wi	2	7	Sp Su	2	7	Sp Su	2		Sp Su	2		Sp Su	2		Sp Su
THUNDER BAY	*3	*2	Sp,Su Fa	*3	*2	Sp,Su Fa	*1	*10	-	*1	*3	Sp	*1	*10	-	-	-	-
WINNIPEG	*2	-	Sp Su	*2	-	Sp Su	1	-	Sp Su	*1	-	Sp Su	-	*15	None Obs.	-	-	None Obs.
REGINA	*1	1	Sp	*3	1	Sp	-	-	-	*1	1	Sp	-	-	-	-	-	-
CALGARY	-	-	Sp,Su	-	-	Sp,Su	-	-	Sp,Su	-	-	Sp,Su	-	-	Sp,Su	-	-	Sp,Su
VANCOUVER	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

NOTES: - = Information not given
* = Information given is considered acceptable

SECTION: 4 Ponding in Residential Areas	WATER PONDS IN BACK YARDS			WATER PONDS AT INLETS			WATER ENTERS BASEMENTS			WATER JUST COVERS STREET			WATER RISES ABOVE CURBS			MANHOLE COVER POPS OFF		
	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year	Times per Year	Years Between Occurrences	Period of Year
ST. JOHN'S	-	None Obs.	-	4	-	Sp,Fa Wi	3	*5	Sp,Fa Wi	3	*2	Sp,Fa Wi	3	*5	Sp,Fa Wi	3	*5	Sp,Fa Wi
BATHURST	2	2	Su	3	2	Sp,Su Wi	1	5	Sp Su	1	3	Su	1	5	Su	-	None Obs.	-
FREDERICTON	*4	2	*Sp	4	2	*Sp	*1	2	*Sp	4	2	*Sp	*1	2	*Sp	*1	2	*Sp
ANJOU	-	None Obs.	-	-	None Obs.	-	2	-	Su	-	None Obs.	-	-	None Obs.	-	-	None Obs.	-
CHICOUTIMI	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	2	Su
SAINTE-FOY	2	-	-	*1	-	-	4	-	-	2	-	-	2	-	-	4	-	-
SHERBROOKE	-	-	-	-	-	-	-	3	Su	-	-	-	-	-	-	-	-	-
CAP-DE-LA-MADELEINE	-	-	-	-	-	-	*3	*3	Su	2	*2	Sp	*1	*5	Su	2	*10	Su
VALLEYFIELD	*3	*2	*Sp	*3	*2	*Sp	*3	*2	*Sp	*3	*2	*Sp	*3	*2	*Sp	*3	*2	*Sp
SAINT LAURENT	*1	*3	Sp	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
HULL	*1	-	Sp	1	*5	Sp,Su	2	*5	Su	2	*5	Su	2	*5	Su	2	*5	Su
PIERREFONDS	1	3	Sp	1	3	Sp	1	3	Sp	*1	5	Sp	*1	5	Sp	1	5	Sp
KINGSTON	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
NORTH BAY	4	-	Sp Su	*1	-	Su	*2	-	Sp	*1	-	Su	-	None Obs.	-	-	None Obs.	-
SUDBURY	4/*1	-	Sp Su	4/*1	-	Sp Su	-	-	Sp	1	*2	Sp,Su* Fa	-	*2	-	-	Sp	-
TIMMINS	*1	*3	Su	*1	*3	Su	*1	*3	Su	*1	*3	Su	*1	*3	Su	*1	*3	Su
GUELPH	-	-	-	2	2	Sp,Fa	-	-	-	-	-	-	-	-	-	-	-	-
BRANTFORD	-	-	-	-	-	-	-	-	-	*1	-	Sp	*1	-	Sp	*1	-	Sp
WATERLOO	2	-	Sp	1	-	Sp	3	-	Sp	-	3	Su	-	10	Su	-	None Obs.	-
SARNIA	-	2	-	-	2	-	-	-	-	-	-	-	-	-	-	-	-	-

* Information given is considered acceptable

SECTION: 5,6,7 5) Maintenance Problems 6) Sewer Materials 7) Inlets and Gutters	(5) MAINTENANCE PROBLEMS							(6) MATERIALS USED FOR STORM SEWER CONSTRUCTION (APPROX. %)				(7) INLETS AND GUTTERS					
	General Cleaning of Grates & Inlets	Collapsed Storm Sewers - Rebuild Catch Basins, etc.	Children or Animals	Flooding Intersections & Basements	Clean Storm Sewers	Freezing	Roots	Concrete Pipe	Corrugated Steel Pipe	Asbestos Cement Pipe	Other	Depressed Inlets	Approx. % Dep. Inlets	Average Inlet Depression	Average Inlet Length	Max. Width Gutter Flow	Average Inlet Spacing
HALIFAX	B	N	I	N	I	N	I	100	-		-	No	-	-	2.5'	1.5'	250'
LAVAL	I	I	-	I	I	I	I	100	-	-	-	Yes	-	3"	2.5'	-	175'
QUEBEC CITY	N	I	I	N	N	I	I	90	-	-	10 Clay	Yes	-	1"	2-3'	2.0'	150'
MONTREAL	N	I	I	N	N	I	I	70	-	-	5 cast in place	No	-	-	1.5'	No Gutt.	200'
OTTAWA	B	N	I	B	N	B	N	94	5	1	-	No	-	-	2'	0.5'	150'
SCARBOROUGH	N	N	-	-	N	-	N	92	8	-	-	Yes	-	1"	2'	1'	300'
TORONTO	N	I	I	I	N	I	I	94	-	1	4 Cast in place 1 Vitrif.	Dished	-	3/4"	2'	2'	150-200'
MISSISSAUGA	N	I	N	I	N	I	I	95	5	-	-	-	-	-	-	-	-
ST. CATHARINES	N	I	I	N		I	N	95	5	-	-	No	-	-	2'	1'	Varies
WINDSOR	N	N	I	B	-	I	N	100	-	-	-	No	-	-	2'	1'	300'
HAMILTON	N	I	I	B	I	I	N	75	0.1	0.1	24.8 Clay & Brick	No	-	-	2'	2'	250-300'
KITCHENER	N	B	I	I	I	I	N	90	10	-	-	Yes	-	3/4"	2'	2'	300'
THUNDER BAY	B	N	I	N	N	I	I	Yes	-	Yes	-	Yes	-	2"	1.5'	-	300'
WINNIPEG	N	B	N	I	I	N	B	98	2	-	-	Yes	20	2"	1.25'	3'-4'	400'-500'
REGINA	N	I	I	I	N	I	I	90	-	-	Clay	No	-	-	1.5'	1.5'	300'
CALGARY	N	I	N	I	I	B	I	99	0.5	-	0.5 Clay, Iron	-	-	-	-	-	-
VANCOUVER REGION	N	-	-	-	N	-	-	Yes	-	-	Cast.Reinf. Concrete	-	-	-	-	-	-

NOTE: Problem was: B = Biggest; N = Normal; I = Infrequent

SECTIONS: 5,6,7

SECTIONS: 5,6,7 5) Maintenance Problems 6) Sewer Materials 7) Inlets and Gutters	(5) MAINTENANCE PROBLEMS								(6) MATERIALS USED FOR STORM SEWER CONSTRUCTION (APPROX. %)				(7) INLETS AND GUTTERS					
	General Cleaning Grates & Inlets	Collapsed Storm Sewers - Rebuild Catch Basins, etc.	Children - Animals	Flooding Intersections & Basements	Clean Storm Sewers	Freezing	Roots	Comments	Concrete Pipe	Corrugated Steel Pipe	Asbestos Cement Pipe	Other	Depressed Inlets	Approx. # Dep. Inlets	Average Inlet Depression	Average Inlet Length	Max. Width Gutter Flow	Average Inlet Spacing
ST. JOHN'S	B	I	N	B	I	I	I	Flooding from Blockage of Catch Basin Leads	100	-	-	Will soon specify corr. steel	No	-	-	2'	2'	400'
BATHURST	B	I	N	I	I	B	I	Freezing Catch basins & Driveway Culverts	90	10	-	Corrugated steel for drvwy culv.	-	25	4"	30'	3'	-
FREDERICTON	N	N	I	B	N	N	I		99	1	-	Corrugat.for lgr sizes & outfalls	-	-	-	-	1.5'	300'
ANJOU	B	B	N	I	I	N	I		100	-	-		Yes	-	2"	2.5'	-	175'
CHICOUTIMI	I	-	-	I	N	-	-		90	10	-		-	-	2"	2'		250'
SAINTE-FOY	N	I	-	N	I	I	I		95	5	-		Yes	-	2"	2.5'	-	250'
SHERBROOKE	B	N	-	N	N	I	B		100	-	-	Planning use of plastic pipe	No	-	-	2'	-	-
CAP-DE-LA-MADELEINE	N	N	I	N	I	I	B		98	2	-		No	-	-	-	-	300'
VALLEYFIELD	N	I	I	N	N	N	I		100	-	-		Yes	-	1.5"	50'	2'	100'
SAINT LAURENT	I	N	I	I	I	B	N		60	5	35		Yes	-	.5"	2.5'	-	150'
HULL	B	N	N	B I	-	B	I		90	2	-	Cast in place Reinf. concrete	No	5	-	-	Nil	200'
PIERREFONDS	N	I	I	N	N	B	I		95	-	5		Yes	-	3"	1'	2.5'	200'
KINGSTON	N	I	N	N	I	N	N	Spring/fall cleaning leaves from catch basins	Specified	-	-		No	-	Var.	2'	-	300'
NORTH BAY	B	N	-	I	-	-	-		80	15	5	Corrugated used off-street	No	-	-	400'	1.5'	275'
SUDBURY	B	I	I	I	I	I	I	Many residential areas drained by open ditches leading to minor flooding due to frozen culverts in spring.	90	5	5	Some old sewers Vitrified clay	No	-	-	-	1.0'	250'
TIMMINS	B	I	N	I	B	N	I		100	-	-		Yes	-	3"	-	1.0'	200' 300'
GUELPH	B	I	I	B	I	I	I		95	5	-		No	-	-	2'	1.5'	200'
BRANTFORD	N	I	I	I	N	-	I		90	-	-	10 Vitrif.clay	No	-	-	2'	2'	200'
WATERLOO	N	N	I	I	N	I	I		90	10	-		No	-	-	2'	1.5'	300'
SARNIA	I	I	I	I	I	N	N		99	1			No	-	-	2	1.5'	300'

NOTE: Problem was: B = Biggest; N = Normal; I = Infrequent

SECTIONS: 8, 9

8) Method of Snow Removal 9) Problems Associated with Freezing	(8) METHOD OF SNOW DISPOSAL			(9) PROBLEMS ASSOCIATED WITH FREEZING	
	Dump into Lake	Dump into River	Other	Description	Est. Cost Per Year
HALIFAX	-	-	Dumped into Halifax Harbour	Heavy rain after snow plugs catch basins	$700,000
LAVAL	-	-	Land disposal	No problems	-
QUEBEC CITY	-	St. Lawrence	2 Snow melters	Frozen catch basins	$ 25,000
MONTREAL	-	St. Lawrence Des Prairies	4 Snow melters Land disposal Collector sewers	Catch basins	$ 50,000
OTTAWA	-	-	Land disposal	Flooding caused by frozen culverts, ditches, catch basins	$ 50,000
SCARBOROUGH	-	-	Land disposal sites which drain to watercourses or storm sewers	Catch basins and driveway culverts	$ 50,000
TORONTO	Lake Ontario in emergency	-	Land disposal	No problems	-
MISSISSAUGA	-	-	Land disposal	Freezing catch basins	$5,000-$6,000
ST. CATHARINES	-	Some melts into 12-Mile Creek	Land disposal	No problems	-
WINDSOR	-	Detroit River in emergency	Infrequent removal Use of salt melting	Increased runoff during thaw	-
HAMILTON	-	-	Land disposal	-	-
KITCHENER	-	-	Land disposal and sanitary sewer	-	-
THUNDER BAY	-	-	Land disposal	Catch basins & culverts	$ 25,000
WINNIPEG	-	Yes	-	-	-
REGINA	-	-	Land disposal	Catch basins	$ 10,000
CALGARY	-	Yes	-	Catch basins and shallow mains	$ 52,000
VANCOUVER REGION	-	-	-	No problems	-

SECTIONS: 8,9

SECTIONS: 8, 9

8) Method of Snow Removal / 9) Problems Associated with Freezing	(8) METHOD OF SNOW DISPOSAL			(9) PROBLEMS ASSOCIATED WITH FREEZING	
	Dump into Lake	Dump into River	Other	Description	Est. Cost Per Year
ST. JOHN'S	-	-	Land disposal and Harbour dumping	Freeze/thaw cycles & salt use breaks down asphalt	-
BATHURST	-	-	Bathurst Harbour	Inlet & culvert freezing	$ 4,000
FREDERICTON	-	St. John		Frozen catchbasins, shallow sewers freeze	-
ANJOU	-	-	Land disposal	Calcium chloride to melt ice in inlets	$ 600
CHICOUTIMI		Yes		Salting and sanding inter-sections when snow continues	$ 66,875
SAINTE-FOY	-	-	Land disposal	Melt ice in catchbasins	$ 50,000
SHERBROOKE	-	Yes	Land disp. in quarry	-	-
CAP-DE-LA-MADELEINE	-	Yes-St.Maurice St.Laurent	Land disposal	None	Nil
VALLEYFIELD	-	Canal de Beauharnois	Land disposal	Frozen catch basins - corro-sion by salt dam. to hydrants	$ 10,000
SAINT LAURENT	-	-	Land disposal	None	-
HULL	-	-	Land disposal	Blocked catch basins	-
PIERREFONDS	-	Yes	Land disposal	None	$100,000
KINGSTON	-	-	Land disposal	Ice-blocked catch basins melted by calcium chloride	-
NORTH BAY	-	-	Land disposal	Freezing of inlets & driveway culverts	$ 35,000
SUDBURY	-	-	Land disposal	Frozen sewer inlets & driveway culverts	$100,000
TIMMINS	-	-	Land disposal	Clean and thaw inlets, ditches & catch basins	-
GUELPH	-	-	Land disposal adj.to drain.courses	Snow blocking culverts and catch basins	$ 3,600
BRANTFORD	-	-	Land disposal	Very infrequent	-
WATERLOO	-	-	Land disposal	Snow and ice removal from catch basin grates	-
SARNIA	-	St. Clair (Heavy snowfall 1/5 yrs)	Land disposal	Thawing of catch basins and culverts	-

SECTIONS: 10 & 11

10) Stormwater Drainage Design Criteria 11) Standards	(10) STORMWATER DRAINAGE DESIGN CRITERIA							(11) STANDARDS			
	Adopted Criteria	Criteria Require Dev. runoff no greater than natural	Specified Runoff Rate	Copies of Criteria Available	Criteria being developed	Do criteria require on-site detention	Mandatory to satisfy crit-eria before construction	Design Using ASCE	Specific Standards	Safety Standards	Other
HALIFAX	Yes	No	No	Yes	Yes	Yes	Yes	-	Yes	-	Flat roofs have controlled drains
LAVAL	No	No	-	No	No	No	No	Yes	-	-	-
QUEBEC CITY	Yes	No	-	Yes	No	No	Yes	-	Yes	-	-
MONTREAL	Yes	No	Yes	Yes	No	No	Yes	-	Yes	Municipal & Provincial	-
OTTAWA	Yes	No	Yes	Yes	No	No	No	No	Yes	Dept. Labour	City of Ottawa Design Manual
SCARBOROUGH	Yes	No	-	Yes	-	No	Yes	-	Yes	-	-
TORONTO	Yes	No	No	Yes	-	No	No	-	Yes	-	-
MISSISSAUGA	Yes	No	No	Yes	-	No	Yes	-	Yes	Dept.Labour Inlet & Out-let Gratings	-
ST. CATHARINES	Yes	No	No	Yes	Yes	Yes	Yes	Yes	-	-	-
WINDSOR	Yes	No	No	Yes	No	No	Yes	Yes	-	-	-
HAMILTON	Yes	No	No	Yes	-	No	Yes	-	-	-	City Standards
KITCHENER	Yes	No	-	Yes	No	No	Yes	Yes	-	-	-
THUNDER BAY	Yes	No	No	Yes	-	No	Yes	-	Yes	-	-
WINNIPEG	No	No	No	No	Yes	Yes	Yes	-	-	-	None developed yet
REGINA	Yes	No	No	Yes	-	No	Yes	Yes	-	-	-
CALGARY	Yes	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Workmens Compensation Board	Under review
VANCOUVER REGION	–	No	No	No	Yes	No	-	Yes	-	-	-

SECTIONS: 10, 11 10) Stormwater Drainage Design Criteria 11) Standards	(10) STORMWATER DRAINAGE DESIGN CRITERIA							(11) STANDARDS			
	Adopted Criteria	Criteria Require Dev. runoff no greater than natural	Specified Runoff Rate	Copies of Criteria Available	Criteria being developed	Criteria require onsite detention	Mandatory to satisfy criteria before construction	Design Using ASCE	Specific Standards	Safety Standards	Other
ST. JOHN'S	Yes	No	No	Yes	No	No	Yes	-	-	-	Design using rational method
BATHURST	Yes	No	Yes	Yes	-	No	No	-	Yes	-	Specific standards prepared by consultants
FREDERICTON	-	-	-	No	Yes	-	No	Yes			
ANJOU	Yes	No	No	-	No	No	Yes	-	-	-	
CHICOUTIMI	Yes	Yes	Yes	Yes	No	Yes	Yes	-	Yes	-	Max.storm 1.5"/hr or $1.5'^3$/Sec/acre
SAINTE-FOY	Yes	No	No	Yes	No	No	Yes	-	-	National Building Code	
SHERBROOKE	No	-	-	-	-	-	-	Yes	-	-	
CAP-DE-LA-MADELEINE	Yes	No	No	No	No	No	Yes	-	-		Based on City of Montreal Rate 1 in 5 yrs. 20% runoff
VALLEYFIELD	Yes	Yes	Yes	Yes	Yes	No	No				City of Mont. standards
SAINT LAURENT	Yes	No	Yes	Yes	No	No	Yes	No	Yes	-	
HULL	Yes	-	-	Yes	No	No	Yes	Yes	-	-	Quebec water management
PIERREFONDS	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes		Yes	Based on ASCE Standards
KINGSTON	Yes	No	Yes	Yes	No	No	No	Yes	-	-	-
NORTH BAY	Yes	No	Yes	Yes	-	No	No				Standards have been designed by municipality
SUDBURY	Yes	No	No	Yes	No	No	No	No	Yes	-	Design acc'd to city stan
TIMMINS	Yes	No	No	Yes	-	No	Yes	-	Yes	-	-
GUELPH	Yes	No	Yes	No	Yes	Yes	Yes	Yes	Yes	Yes	-
BRANTFORD	Yes	No	No	Yes	No	No	Yes	-	Yes	-	-
WATERLOO	Yes	No	No	Yes	-	No	No	-	Yes	-	-
SARNIA	Yes	No	Yes	Yes	Yes	No	Yes	-	Yes		Standards designed by consultants

SECTIONS: 12 & 13

12) Methods for Runoff Computations 13) Design Storm Frequency (Yrs)	(12) METHODS FOR RUNOFF COMPUTATIONS				(13) DESIGN STORM FREQUENCY (YRS.)						
					SEWERS		CULVERTS				
	Rational Method	Computations by Hand	Computations by Hand & Computer	Other Methods	Residential	Commercial & Industrial	Driveways	Minor Roads	Major Roads	Freeways	Other
HALIFAX	X	X			5	5	5	5	5	10	
LAVAL	X				10	5	-	-	-	-	
QUEBEC CITY	X	X			5	5	5	5	-	-	
MONTREAL	X		X	Hydrograph Method	10	10	10	10	10	10	
OTTAWA	X	X			5-25	10-25	Designed for existing flow				
SCARBOROUGH	X	X	Some		-	-	10	20	20	-	
TORONTO	X		X		2	5	Not Used	2	2-5	-	
MISSISSAUGA	X	X			10	10	-	-	-	-	Culverts 25 yr.
ST. CATHARINES	X	X			5	5	5	5	5	-	
WINDSOR	X	X			5	5	5	5	-	-	
HAMILTON	X		X		-	-	-	-	-	-	Special rainfall adapted by city - varies 10-25 yrs
KITCHENER	X	X			5	5	-	-	-	-	
THUNDER BAY	X	X			2	2	2	2	2	-	
WINNIPEG	X	X	X	Chicago Hydrograph	5	5-10	5	5-10	50	50	
REGINA	X	X			5	5-10	-	-	-	-	
CALGARY	X	X			5	5	5	5	5	5	10-yr. for underpasses (pumped)
VANCOUVER REGION	X	X			10	10	-	-	10	-	

SECTIONS: 12, 13 12) Methods for Runoff Computations 13) Design Storm Frequency (Yrs)	(12) METHODS FOR RUNOFF COMPUTATIONS				(13) DESIGN STORM FREQUENCY (YRS.)						
					SEWERS		CULVERTS				
	Rational Method	Computations by Hand	Computations by Hand & Computer	Other Methods	Residential	Commercial & Industrial	Driveways	Minor Roads	Major Roads	Freeways	Other
ST. JOHN'S	X	X			5	5	5	5	5	-	
BATHURST	X	X			5	5	5	5	5	-	
FREDERICTON	X	X			5	10-15	-	-	-	-	
ANJOU	X	X			-	-	-	-	-	-	
CHICOUTIMI	X	X			-	-	-	-	-	-	
SAINTE-FOY		X		Emperical-McMath Formula	25	25	-	-	25	-	
SHERBROOKE	X	X			5	-	-	-	-	-	
CAP-DE-LA-MADELEINE		X			5	5	-	-	-	-	
VALLEYFIELD	X	X		Montreal	5	5	-	-	-	-	
SAINT LAURENT	X	X			5	5	-	-	-	-	
HULL	X	X			5	15-25	-	-	-	-	
PIERREFONDS	X	X			5	5	5	5	5	5	
KINGSTON	X	X			2	2	2	2	2	2	
NORTH BAY	X	X			10	10	5	5	20	-	
SUDBURY	X	X			2	5	2	2	2	-	
TIMMINS	X	X			2	2	-	-	-	-	
GUELPH	X	X			5	5	5	5	5	-	25-year storm for areas greater than 500 acres
BRANTFORD	X	X			5	5	5	5	-	-	
WATERLOO	X	X			5	5	5	5	25	50	Creek improvements Design 1 in 100 years
SARNIA	X	X			1	2	-	-	-	-	

SECTION: 14 Master Drainage Design	Major & Minor Systems Clearly Defined	Flows Less than 1/50 Frequency Used	Compare Open Channel and Closed Conduits	Selection on Economic Basis	Other Criteria	Degree of Ponding Permissible on Minor System: Sev. Times a Year	Once a Year	Once in several yrs.	System Observed Under Design Storm	Protection Satisfactory under Design Conditons	Occasional Flooding Considered when Design calls for Intermed. Pipe	Cost Estimate for Residential Storm Sewers
HALIFAX	Yes	No	Yes	Yes	5-Yr Storm Associated Runoff			X	Yes	No	Yes	$40/ft. and up depending on pipe size
LAVAL	Yes	Yes	-	-	-			X	No	Yes	No	-
QUEBEC CITY	Yes	Yes	-	Yes	Rainfall Frequency			X	Yes	Yes	No	$15-50/ft. depending on pipe size
MONTREAL	Yes	No	No	Yes	Plans to separate Storm & Sanitary			X	Yes	Yes	No	Combined = $30.80/ft assessable frontage
OTTAWA	Yes	No	Yes	Yes	Aesthetics			X	Yes	Yes	No	-
SCARBOROUGH	Yes	-	Yes	Yes	-			X	Yes	Yes	No	-
TORONTO	Yes	No	No	-	-			X	-	Yes	No	Net Cost $4390/acre
MISSISSAUGA	-	-	Yes	Yes	Conserve Natural Watercourses			X	-	-	No	Storm $29 per ft.road
ST. CATHARINES	No	No	No	-	-			X	No	Yes	No	-
WINDSOR	Yes	-	Yes	-	-			X	-	-	No	-
HAMILTON	Yes	-	Yes	Yes	-			X	Yes	Yes	No	$30 per ft. frontage
KITCHENER	-	-	-	-	-				-	-	Yes	-
THUNDER BAY	-	-	-	-	-				-	-	-	$8/ft. frontage - main $275/home-weeping tile storm sewer connection
WINNIPEG	Yes	Yes	Yes	Yes	Maintenance Aesthetics			X	No	Yes	Yes	$4,000/acre includes trunk
REGINA	-	-	-	-	-				-	-	-	-
CALGARY	Yes	-	Yes	-	-				Yes	Yes	No	-
VANCOUVER REGION	-	No	Yes	-	-				Yes	Yes	No	-

SECTION: 14 Master Drainage Design	Major & Minor Systems Clearly Defined	Flows Less than 1/50 Frequency Used	Compare Open Channel and Closed Conduits	Selection on Economic Basis	Other Criteria	Degree of Ponding Permissible on Minor System: Sev. Times a Year	Degree of Ponding Permissible on Minor System: Once a Year	Degree of Ponding Permissible on Minor System: Once in several years	System Observed Under Design Storm	Protection Satisfactory Under Design Conditions	Occas. Flooding Considered when Design calls for Intermed. Pipe Size	Cost estimate for Residential Storm Sewers
ST. JOHN'S	No	No	No	-	-			X	No	Yes	No	$10-15 per foot
BATHURST	Yes	-	No	-	-			X	No	-	Yes	Not available
FREDERICTON	-	-	-	-	-	-	-	-	-	-	-	-
ANJOU	Yes	No	Yes	-	-			X	-	-	No	-
CHICOUTIMI	Yes	-	-	Yes	-			X	Yes	Yes	No	-
SAINTE-FOY	No	No	-	Yes	-			X	No	-	No	$11 per foot
SHERBROOKE	Yes	-	-	-	-	-	-	-	-	-	No	-
CAP-DE-LA-MADELEINE	No	No	-	-	-	X			Yes	Yes	No	-
VALLEYFIELD	Yes	No	No	Yes	-		X		No	Yes	No	$60 per foot
SAINT LAURENT	Yes	No	-	Yes	-			X	Yes	Yes	No	-
HULL	Yes	No	No	-	-			X	Yes	Yes	No	$20 per foot
PIERREFONDS	Yes	No	Yes	Yes	Yes		X		Yes	Yes	No	$4 Million
KINGSTON	No	-	-	-	-	-	-	-	Yes	Yes	Yes	Fairly high; varies due to high rock format
NORTH BAY	-	-	-	-	-	-	-	-	No	-	-	$30 per foot
SUDBURY	Yes	No	Yes	Yes	-		X		No	-	Yes	-
TIMMINS	No	-	Yes	Yes	-	-	-	-	Yes	Yes	No	-
GUELPH	No	No	Yes	Yes	-			X	No	Yes	No	$25-30 per foot
BRANTFORD	Yes	No	No	-	-			X	No	-	No	$15 per foot
WATERLOO	-	-	-	-	-	-	-	-	Yes	Yes	No	$10 per foot
SARNIA	Yes	No	Yes	-	-			X	No	Yes	No	-

SECTION: 15 Detention	Detention Used for Reduction of Peak Flows	Detention Required	Owner of Detention Facility: Agency	Private	Other	Estimate of Number of Facilities Now Used: Roof top	Parking Lot	Park	Schoolground	Other
HALIFAX	Yes	Yes		Yes		20	5	1	-	-
LAVAL	No	No								
QUEBEC CITY	No	Yes								
MONTREAL	No	Yes								
OTTAWA	No	Yes				Nil	Nil	Nil	Nil	Nil
SCARBOROUGH	No	Yes		Yes		Some				
TORONTO	No	No								
MISSISSAUGA	Yes	Yes			Controlled Roof drains Industrial	Sev. constructed in last two years		-	-	-
ST. CATHARINES	No	Yes								
WINDSOR	-	-								
HAMILTON	No	No				Nil	Nil	Nil	Nil	Nil
KITCHENER	No	No								
THUNDER BAY	No	No								
WINNIPEG	Yes	Yes	Yes							9 Ponds
REGINA	No	No								
CALGARY	Yes	-	Yes	Yes	-	Data not available				
VANCOUVER REGION	Yes	-	Yes		Municipality	-	-	-	-	Reservoirs

SECTION: 15 Detention	Detention Used for Peak Flow Reduction	Detention Required	Owner of Detention Facility			Estimate of Number of Facilities Now Used				
			Agency	Private	Other	Roof top	Parking Lot	Park	Schoolground	Other
ST. JOHN'S	No	No	-	-	-	-	-	-	-	-
BATHURST	No	No	-	-	-	-	-	-	-	-
FREDERICTON	No	No	-	-	-	-	-	-	-	-
ANJOU	No	No	-	-	-	-	-	-	-	-
CHICOUTIMI	Yes	Yes	-	-	Streets	-	-	-	-	-
SAINTE-FOY	No	No	-	-	-	-	-	-	-	-
SHERBROOKE	No	-	-	Yes	-	25	5	-	-	-
CAP-DE-LA-MADELEINE	No	Yes	-	-	-	-	-	-	-	Enclosed lands
VALLEYFIELD	Yes	Yes	City Pipes	-	-	-	-	-	-	-
SAINT LAURENT	Yes	Yes	-	Yes	-	5	3	-	-	-
HULL	No	Yes	-	-	-	-	-	-	-	-
PIERREFONDS	No	No	-	-	-	-	-	-	-	-
KINGSTON	No	No	-	-	-	-	-	-	-	-
NORTH BAY	No	Yes	-	-	-	-	-	-	-	-
SUDBURY	No	Yes	-	-	-	-	-	-	-	-
TIMMINS	No	No	-	-	-	-	-	-	-	-
GUELPH	No	Yes	-	-	-	-	-	-	-	-
BRANTFORD	No	No	-	-	-	-	-	-	-	-
WATERLOO	No	-	Grand R.C.A.	-	-	-	-	-	-	-
SARNIA	No	No	-	-	-	-	-	-	-	-

SECTION: 16 Storm Sewer Use Regulations	Roof drain Discharge				Foundation Drain Connection			Inlet Design											
	Onto Property	Into Gutters	Sewers or Ditches	Other	Sanitary Sewers	Storm Sewers	Combined	Max. Catch Basin Spacing (Ft.)	Min. Sewer Size (In.)	Min. Sewer Cover (Ft.)	Max. Drainage Distance to First Inlet (Ft.)	ASCE Manual No. 9	Other	Sewer Distance From C_L Road (Ft.)	Max. Distance Between Manholes	Prop. Catch Basins in New Development	Double House Connections Allowed	Bend & Manhole Losses Considered in Design
HALIFAX	A		A	Comb. Sewer		A	C	350	12	7	200	X		5	300	Yes	No	Yes
LAVAL			C		C		C	150	12	-	-	X		-	400	Yes	Yes	Yes
QUEBEC CITY	C	C	C		A	A	C	100-200	6	-	25		Emp.	0	300-500	Yes	Yes	-
MONTREAL			C	C		C	C	250	18	9	250		Emp.	0	300	Yes	No	No
OTTAWA	C		A		B	C	C	150	12	7	-		Varies	10	360	Yes	No	Yes
SCARBOROUGH			C			C	C	300	12	7	-	X		5	300	Yes	Yes	Yes
TORONTO				Not Sanitary	C	A	B	Varies	12	6	200	X		0	300	Yes	Dis-cour-aged	No
MISSISSAUGA	B		C			C		300	10	4	300			5	300	Yes	Yes	Yes
ST. CATHARINES			C		C			300	10	3.5	-		None	6	330	Yes	Yes	No
WINDSOR			C		A	A	A	300	12	3.5	-	X		Varies	400	Yes	No	No
HAMILTON			C			A	B	250-300	12	9	250		Stan. Size	1.5	300-500	Yes	No	Yes
KITCHENER	C	C	C		C	C		300	12	4	-			8	300	Yes	No	Yes
THUNDER BAY	C				B	A		300	12	5	-			19	360	Yes	No	Yes
WINNIPEG	A		B		C		C	400-500	10	5	250-300	X	Seelye	14-19	400	Yes	No	Yes
REGINA		A			A	A		350	8	6	350	X		-	400	Yes	Yes	No
CALGARY			A			A		1000	12	4	1000			18-36	550	Yes	No	Yes
VANCOUVER REGION			A			A	A	-	-	-	-			-	-	-	-	-

NOTE: A = Present Policy B = Policy Prevailing last 25 years C = Present Policy and Prevailing Policy for Last 25 Years

SECTION: 16 Storm Sewer Use Regulations	Roof drain Discharge			Foundation Drain Connection				Inlet Design										
	Onto Property	Into Gutters	Sewers or Ditches	Sanitary Sewers	Storm Sewers	Combined	Other	Max. Catch Basin Spacing (Ft.)	Min. Sewer Size (In.)	Min. Sewer Cover (Ft.)	Max. Drainage Distance to First Inlet (Ft.)	ASCE Manual No. 9	Other	Sewer Distance From ℄ Road (Ft.)	Max. Distance Between Manholes	Prop. Catch Basins in New Development	Double House Connections Allowed	Bend & Manhole Losses Considered in Design
ST. JOHN'S	C		C		C			450	12	3	5280	-	-	1/2 Dist. ℄ to curb	450	Yes	Yes	No
BATHURST			C		C			300	12	2	-	-	-	Under curb	-	No	No	Yes
FREDERICTON	C	C	C	B	C	B		300	10	6	100	-	-	14	300	Yes	-	No
ANJOU	A		A		A	A		300	15	-	-	-	-	10	400	Yes	No	-
CHICOUTIMI	B		A		A			250	8	-	250	-	-	0	500	Yes	Yes	No
SAINTE-FOY	C			C				300	12	2	250	-	-	0	500	Yes	Yes	No
SHERBROOKE					A	A		500	6	-	500	Yes	-	27-43	400	-	Yes	No
CAP-DE-LA-MADELEINE		B	A		A	B		250	12	2	250	Yes	-	5-10	400	Yes	Yes	No
VALLEYFIELD			C		C			100	15	4	500	-	Mfgs Stand	20-30	250	Yes	No	Yes
SAINT LAURENT			C		C	C		150	10	-	250	-	-	0	300-350	Yes	Yes	No
HULL	B		A		A	B		200	12	0.75	6000	-	-	Varies	250	Yes	No	Yes
PIERREFONDS	B	B	A	B	A			200	15	2.5	15	Yes	-	8	350	Yes	No	Yes
KINGSTON	A	A	A	B	A			300	12	2.5	400	Yes	-	0-2	400	Yes	Yes	Yes
NORTH BAY	A		A		A			300	12	2	300	-	None	10	300	Yes	Yes	No
SUDBURY	C	C	C	C				300	12	6	Varies	-	-	5	300-500	Yes	No	No
TIMMINS	A		A	B	A			300	12	5	-	-	-	12-25	300	Yes	No	Yes
GUELPH			A		A			300	12	6	-	Yes	-	10	300	Yes	Yes	Yes
BRANTFORD	A				A			250	12	5	250	-	-	10	375	Yes	No	Yes
WATERLOO	C	C	C	C				325	12	4	300	-	-	Curb	350	Yes	-	No
SARNIA	A		A		A	A		300	10	5	-	Yes	-	9	350-700	Yes	No	Yes

NOTE: A = Present policy B = Policy Prevailing last 25 years C = Present Policy and Policy Prevailing for last 25 years

SECTION: 17 Application of Rational Method	Inlet Time (Min.)		Runoff Coefficient for Various Land Uses * Residential														Design Velocities (FPS)					
																	Earth Channel		Lined Channel		Pipe	
	Maximum	Minimum	Single Family	Semi-Detached	Townhouses	Low Density Apartments	High Density Apartments	Grass	Parks & Playgrounds	Industrial	Hospitals, Churches, Schools	Paved Areas & Other Surfaces	Commercial & Downtown	Rat. Formula Used Only for Peak Flows	Rat. Formula Used for Triangular Hydrograph	Inlet Design Method Used	Max.	Min.	Max.	Min.	Max.	Min.
HALIFAX	Var	5	.40	.45		.55			.20	.80	.70	1.0	.85	Yes	-	Yes	-	-	-	-	15-20	3
LAVAL	30	5	.30							.50			.50 .90	Yes	-	-	-	-	-	-	-	-
QUEBEC CITY	10	10	.30 .40					.10		.40 .60		.70 .80	.50 .60	Yes	-	-	-	-	-	-	-	2.5
MONTREAL	10	5	.20 .30						.10	.40 .50			.50 .60	Yes	Yes	No	-	-	-	-	8	2.5
OTTAWA	Var	Var	.35			.50	.80		.05	.40				Yes	-	Yes	-	-	-	-	12	3
SCARBOROUGH	10	5	.45	.60	.70	.75			.20 .25	.75	.75		.90	-	-	Yes	-	-	-	-	15	3
TORONTO	8	-	.70										.90	Yes	-	No	-	2	-	2	-	2
MISSISSAUGA	15-18	-	.35 .40	.35 .40	.40 .45	.55 .60	.40 .45	.10 .20		.60 .75		.85	.75	Yes	-	-	-	-	-	-	-	2.5
ST. CATHARINES	10	None	.40		.60				.20	.70		.90 .95	.70	Yes	-	No	-	-	-	-	15	2.5
WINDSOR	20	20						.15 .20	.20			.90 .95		-	-	No	-	-	-	-	-	2.5
HAMILTON	10	5	.40	.40	.50 .60					.50 .60			.90	Yes	-	No	-	-	-	-	10	3
KITCHENER	-	15	.40		.70				.25	.75	.75		.90	No	-	-	-	-	-	-	12	2.5
THUNDER BAY	-	15	.30		.60				.20 .25	.60 .75			.60 .75	Yes	-	-	3	2	7	2	15	2
WINNIPEG	15	10	.35							.70 .90			.70 .90	Yes	-	Yes	4-5	2	4-5	2	8-10	3
REGINA	-	-	.38							.38			.68	Yes	-	No	3	3	-	-	-	3
CALGARY	-	10	.30						.15	.30 .75			.90	-	-	-	-	3	-	3	-	3
VANCOUVER REGION	15	5	.36 .84							.36 .84			.36 .84	Yes	No	Yes	-	-	-	-	-	-

* NOTE: Those questionnaires returning "Residential" were assigned to the Single-Family column

SECTION: 17 Application of Rational Method	Inlet Time (Min.) Maximum	Inlet Time (Min.) Minimum	Runoff Coefficient for Various Land Uses: Single Family *	Semi-Detached	Townhouses	Low Density Apartments	High Density Apartments	Bare Ground	Grass	Parks & Playgrounds	Industrial	Hospitals, Churches, etc.	Paved Areas & Other Surfaces	Commercial & Downtown	Rat. Formula Used Only for Peak Flows	Rat. Formula Used for Triangular Hydrograph	Inlet Design Method Used	Design Velocities (FPS): Earth Channel Max.	Earth Channel Min.	Lined Channel Max.	Lined Channel Min.	Pipe Max.	Pipe Min.
ST. JOHN'S	40	6	-	-	-	-	-	-	-	-	-	-	-	-	Yes	-	No	-	-	-	-	-	2.5
BATHURST	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Designed by Consultants					-
FREDERICTON	30	5	-	-	-	-	-	-	-	-	-	-	-	-	Yes	-	-	-	-	-	-	-	-
ANJOU	15	5	-	-	-	-	-	-	-	-	-	-	-	-	Yes	-	No	-	-	-	-	-	-
CHICOUTIMI	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
SAINTE-FOY	15	5	.50						.35		.50	.40	.90	.90	Yes	Yes	No	-	-	-	-	25.0	2.0
SHERBROOKE	15	5	.50	.60										.80	-	-	No	-	-	-	-	-	-
CAP-DE-LA-MADELEINE	-	10													Yes	-	No	-	-	-	-	-	-
VALLEYFIELD	30	20													Yes	-	No	-	-	25	10	-	-
SAINT LAURENT	-	15	.40 (Mean Coefficient)												Yes	-	No	-	-	-	-	-	-
HULL	15	10							.30				.80		Yes	-	-	3	1	4	2	10	2
PIERREFONDS	30	15	.30 .50		.50 .70		.70 .80			.20 .25			.90	.50 .70	Yes	-	No	-	-	-	-	10	2
KINGSTON	15	10						.30	.20				.90		Yes	-	-	4	2	12	2	15	2
NORTH BAY	None	20	.35	** - .45 -		** - .60 -								.80	Yes	-	No	-	-	-	-	12	2
SUDBURY	15	5	.40								.60			.70	Yes	-	No	-	-	-	-	12	2.5
TIMMINS	20	10	.50								.70			.90	-	-	No	-	-	-	-	-	2.5
GUELPH	-	15	.30 .50	.40 .60	.60 .75	**.50 - **.70 -				.10 .35	.50 .90			.70 .95	Yes	-	No	6	2.5	12	2.5	15	2.5
BRANTFORD	15	15	.30 .40							.15 .25	.30 .75			.50 .85	Yes	-	No	5	2	Critical Velocity		8	2.5
WATERLOO	10	-	.40	.50	.60					.20	.70			.80	Yes	-	-	-	-	-	-	Varies	3
SARNIA	15	5	.45		.60					.15 .20	.60			.85	Yes	-	Yes	4	1.2	-	-	7	2.1

** Combined Coefficient

* Note: Responses to questionnaires returning "Residential" were assigned to single family

SECTIONS: 18,19 18) Characteristic Storm Data 19) Attitudes, Environmental Aspects Considered	(18) CHARACTERISTIC STORM DATA Intensity (In. Per Hour)												(19) ATTITUDES, ENVIRONMENTAL ASPECTS CONSIDERED					
	2 Year				5 Year				10 Year				Changes in Downstream Peak Flow	Changes in Groundwater Regime	Changes in Sediment Load	Impact on Wildlife	Pollution of Receiving Water Body	Comments and Recommended Measures
	5 Min	10 Min	30 Min	60 Min	5 Min	10 Min	30 Min	60 Min	5 Min	10 Min	30 Min	60 Min						
HALIFAX	3.2	2.2	1.2	0.8	4.2	2.7	1.4	0.9	5.0	3.1	1.6	1.1	Yes	Yes	Yes	By Others	Yes	Discharge treated storm waters into lakes
LAVAL	-	-	-	-	-	-	-	-	-	-	-	-	Yes	-	-	-	Yes	
QUEBEC CITY	One historical curve based on 20 years of data								4.9	4.3	2.6	1.8	Yes	-	No	No	Yes	
MONTREAL	3.9	3.0	1.5	0.8	4.6	3.7	1.9	1.1	4.8	3.9	2.3	1.4	Yes	No	Yes	Yes	Yes	Collect. & treat. planned
OTTAWA	-	-	-	-	5.3	4.0	2.1	1.2	5.8	4.6	2.4	1.4	No	Effect on wells	No	No	No	May consider treatment of storm effluents in future
SCARBOROUGH	-	-	-	-	4.6	3.7	2.1	1.3	-	-	-	-	-	-	-	-	Yes	Program underway to eliminate all combined sewers
TORONTO	4.8	3.3	1.8	1.1	6.4	4.3	2.4	1.4	7.5	4.9	2.8	1.6	Yes	-	-	Yes	Yes	-
MISSISSAUGA	4.5	3.3	1.7	1.0	6.2	4.1	2.2	1.3	7.3	4.9	2.6	1.5	-	-	-	-	-	-
ST.CATHERINES	-	-	-	-	-	3.7	2.2	1.5	-	-	-	-	Yes	No	No	No	No	Possibly should store and route flow thru treat.plant
WINDSOR	-	-	-	-	8.3	4.2	2.5	1.6	-	-	-	-	-	-	-	-	Yes	
HAMILTON	* ←					6.0	2.8	1.7				→ *	-	-	-	-	Yes	No undisturbed watersheds within the city
KITCHENER	4.2	3.2	1.7	0.9	4.6	3.7	2.1	1.4	5.7	5.0	3.4	2.1	-	-	-	-	-	-
THUNDER BAY	4.0	2.8	1.4	0.9	5.2	3.6	1.7	1.1	6.0	4.2	2.0	1.2	Yes	Yes	-	-	Yes	-
WINNIPEG	4.6	3.2	1.7	1.2	4.8	3.7	2.2	1.3	5.0	4.2	2.4	1.4	Yes	No	-	No	Yes	None
REGINA	-	-	-	-	4.5	3.5	1.9	1.1	-	-	-	1.3	-	-	-	-	-	-
CALGARY	-	-	-	-	-	-	-	-	-	-	-	-	Yes	Yes	-	Yes	Yes	
VANCOUVER REGION	1.4	1.0	0.6	0.4	1.9	1.4	0.8	0.6	2.4	1.7	1.0	0.7	-	-	-	-	Yes	-

* No return frequencies given

SECTIONS: 18, 19 18) Characteristic Storm Data 19) Attitudes and Environmental Aspects Considered	(18) CHARACTERISTIC STORM DATA Intensity (In./Hr.)												(19) ATTITUDES & ENVIRONMENTAL ASPECTS CONSIDERED					
	2 Year				5 Year				10 Year				Changes in Downstream Peak Flow	Changes in Groundwater Regime	Changes in Sediment Load	Impact on Wildlife	Pollution of Receiving Water Body	Comments and Recommended Measures
	5 Min	10 Min	30 Min	60 Min	5 Min	10 Min	30 Min	60 Min	5 Min	10 Min	30 Min	60 Min						
ST. JOHN'S	-	-	-	-	-	2.4	-	-	-	-	-	-	Yes	-	-	-	Yes	
BATHURST	3.6	2.6	1.9	0.6	4.8	3.7	1.9	0.9	5.4	4.2	2.3	1.2	-	-	-	-	-	
FREDERICTON	3.5	2.5	1.2	1.8	4.5	3.3	1.5	1.9	5.4	4.1	1.8	1.95	Yes	Yes	Yes	-	Yes	
ANJOU	-	-	-	-	-	-	-	Yes	-	-	-	-	-	-	-	-	-	
CHICOUTIMI	-	-	-	Yes	-	-	-	-	-	-	-	-	-	-	-	-	-	
SAINTE-FOY	3.0	2.5	1.3	0.8	-	-	-	-	4.2	3.5	2.0	1.3	No	No	No	No	No	
SHERBROOKE	-	-	-	-	4.4	3.6	2.1	1.2	-	-	-	-	-	-	-	-	-	
CAP-DE-LA-MADELEINE	-	-	-	-	-	Yes	-	-	-	-	-	-	-	-	-	-	Yes	
VALLEYFIELD	Yes	-	-	-	-	-	-	-	-		-	-	Yes	Yes	-	-	Yes	Fix Diameter
SAINT LAURENT	15 Minute Storm 1/5 Years												-	-	-	-	-	
HULL	3.9	3.4	1.8	1.1	4.7	4.2	2.2	1.3	5.2	4.6	2.4	1.4	Yes	No	No	No	No	Sedimentation basins
PIERREFONDS	15 Minute Storm 1/5 Years												No	No	No	No	Yes	
KINGSTON	Yarnells 2 and 5 year Curve												No	No	No	No	No	
NORTH BAY	-	-	-	-	3.0	2.3	1.3	1.0	3.0	3.0	1.7	1.1	No	No	No	No	No	
SUDBURY	-	-	-	Yes	-	-	-	Yes	-	-	-	-	-	-	-	-	-	Not considered at this time.
TIMMINS	3.5	3.0	1.3	0.5	-	-	-	-	-	-	-	-	Yes	No	No	No	Yes	
GUELPH	-	-	-	-	-	-	-	-	-	-	-	-	-	Yes	Yes	Yes	Yes	
BRANTFORD	-	-	-	-	-	-	-	Yes	-	-	-	-	Yes	-	Yes	Yes	Yes	
WATERLOO	4.2	3.2	1.7	0.9	5.0	3.8	2.4	1.3	5.7	4.4	2.8	1.7	-	-	-	-	-	
SARNIA	4.3	3.1	1.7	1.1	5.8	4.1	2.4	1.5	-	-	-	-	-	-	-	-	Yes	

SECTIONS: 20, 21 20) Problems of Concern & Research Needs 21) Comments & Other Problems Related to Major Design Policy Changes	(20) PROBLEMS OF CONCERN AND RESEARCH NEEDS							(21) COMMENTS AND OTHER PROBLEMS RELATED TO MAJOR DESIGN POLICY CHANGES
	New Runoff Models	Optimization Techniques	Improved Land Use	Pollution for Urban Runoff	Detention & Storage	Design Storms	Urban Erosion	
HALIFAX	INT	HP	HP	INT	INT	NN	HP	New services to be sized to handle fully developed lands tributary to the site
LAVAL	INT	NN	NN	INT	NN	INT	NN	None
QUEBEC CITY	INT	INT	-	-	INT	INT	-	Separate storm and sanitary sewers
MONTREAL	INT	INT	NN	INT	INT	INT	INT	Design storms have recently been changed to 10-yr instead of 5-year return frequency
OTTAWA	NN	INT	NN	HP	INT	INT	NN	Guidelines required as to degree of pollution acceptable from various sources. Sewer separation program started in 1964. Since 1961 weeping tile flow goes to storm sewers
SCARBOROUGH	INT	-	-	INT	INT	NN	INT	Lake erosion along Scarborough Bluffs Run-off coefficients increased
TORONTO	INT	INT	NN	HP	NN	INT	INT	Road storm sewers designed for 70% of storm runoff are constructed to relieve surcharged combined sewers
MISSISSAUGA	-	-	-	-	-	-	-	-
ST. CATHARINES	INT	HP	INT	HP	HP	INT	HP	Firm Federal & Provincial policies should be formulated
WINDSOR	INT	INT	INT	HP	HP	HP	INT	More research needed concerning storm drainage systems using detention and storage; some detention rec. provided
HAMILTON	INT	INT	-	HP	NN	INT	NN	No major design policy changes in last few years
KITCHENER	-	-	-	-	-	-	-	-
THUNDER BAY	INT	INT	INT	INT	INT	INT	INT	No storm drainage allowed in sanitary sewer system
WINNIPEG	INT	INT	HP	HP	HP	HP	INT	Will set standards and prepare master drainage plan in the near future
REGINA	NN	INT	-	INT	INT	-	-	-
CALGARY	INT	INT	INT	INT	INT	INT	INT	None
VANCOUVER REGION	INT	-	-	INT	INT	INT	-	Regional hydrology study to develop design criteria is currently under way

NOTE: NN = No Need; INT = Interested; HP = High Priority

SECTIONS: 20, 21
20) Problems of Concern and Research Needs
21) Comments and Other Problems Related to Major Design Policy Changes

	(20) PROBLEMS OF CONCERN AND RESEARCH NEEDS							(21) COMMENTS AND OTHER PROBLEMS RELATED TO MAJOR DESIGN POLICY CHANGES
	New Runoff Models	Optimization Techniques	Improved Land Use	Pollution for Urban Runoff	Detention & Storage	Design Storms	Urban Erosion	
ST. JOHN'S	INT	INT	INT	INT	INT	INT	INT	
BATHURST	-	-	-	-	-	-	-	
FREDERICTON	-	-	INT	INT	-	INT	-	
ANJOU	NN	NN	NN	INT	NN	NN	NN	
CHICOUTIMI	NN	NN	NN	INT	NN	NN	NN	
SAINTE-FOY	INT	INT	INT	INT	HP	INT	INT	Separate storm drainage
SHERBROOKE	-	-	-	-	-	-	-	
CAP-DE-LA-MADELEINE	INT	-	INT	INT	INT	-	NN	
VALLEYFIELD	INT	INT	HP	INT	INT	INT	HP	Separate existing combined sewers Construct separate sewers in new developments
SAINT LAURENT	-	-	INT	INT	INT	INT	NN	None
HULL	INT	HP	HP	HP	HP	INT	INT	Convert to separate sewers in city centre Installing separate sewers in new developments
PIERREFONDS	INT	INT	INT	INT	INT	INT	INT	
KINGSTON	INT	INT	NN	INT	INT	INT	INT	Manual No. 9 Hydraulic Grade Line Method to check capacity Would use computer when British Road Research Lab Method or Hydraulic Volume Method is generalized
NORTH BAY	INT	INT	HP	HP	HP	INT	INT	Separation of sewers
SUDBURY	INT	INT	INT	INT	INT	HP	INT	
TIMMINS	INT	INT	INT	INT	INT	INT	INT	
GUELPH	NN	INT	NN	INT	INT	INT	INT	No major design changes in the last few years
BRANTFORD	INT	INT	INT	INT	INT	INT	INT	
WATERLOO	INT	INT	-	-	-	-	-	
SARNIA	-	-	HP	INT	-	INT	-	Separate combined into storm & sanitary systems

NOTES: NN = No Need; INT = Interested; HP = High Priority

A P P E N D I X I I I

Statistical Measures

APPENDIX III

Statistical Measures

Three statistical measures were chosen to evaluate the accuracy of the hydrographs computed by the models when compared to the entire recorded hydrographs. These are the Correlation Coefficient (R), the Special Correlation Coefficient (Rs) and the Integral Square Error (ISE).

If a linear relationship between two variables, the observed hydrograph, O, and the computed hydrograph, C, is assumed, the corresponding linear correlation coefficient is defined by,

$$R = \frac{N\left(\sum_{i=1}^{N} O_i C_i\right) - \left(\sum_{i=1}^{N} O_i\right)\left(\sum_{i=1}^{N} C_i\right)}{\left\{\left[N \sum_{i=1}^{N} O_i^{2} - \left(\sum_{i=1}^{N} O_i\right)^2\right]\left[N \sum_{i=1}^{N} C_i^{2} - \left(\sum_{i=1}^{N} C_i\right)^2\right]\right\}^{1/2}}$$

Where N is the number of observations of O and C. The linear correlation coefficient (R) has the following properties:

i) $-1 \leq R \leq +1$

ii) The closer the value of R is to either +1 or -1, the better is the agreement between the two variables for the assumed linear relationship.

iii) A value of R closer to zero indicates that the two variables are uncorrelated.

Another measure of agreement between the known variable O and its estimated value C can be defined in terms of the sum of the squares of their derivation, or

$$\sum_{i=1}^{N} (O_1 - C_1)^2$$

This can be rearranged in its computational form to express what

is known as the Special Correlation Coefficient, Rs.

$$R_s = \frac{2 \sum_{i=1}^{N} O_i C_i - \sum_{i=1}^{N} C_i^2}{\sum_{i=1}^{N} O_i^2}$$

The Special Correlation Coefficient (R_s) has the following properties:

i) $R_s \leq +1$

ii) $R_s = +1$ if ($O_i = C_i$ for i=1, N)

iii) $R_s = 0$ if $C_i = 2\,O_i$

By comparing the linear correlation coefficient R and the special correlation coefficient R_s, it may be observed that,

1) The Special Correlation Coefficient R_s is similar to the linear correlation coefficient, R, in that the closer the value of R_s is to +1 the better is the agreement between the observed and the estimated values.
2) The Special Correlation Coefficient does not have the property of invariance under change of scale and location.
3) The distribution of R_s is unknown and hence the test of significance of the value of R_s cannot be performed. Thus, although R_s is not a correlation coefficient in the usual sense, it still can be used as a measure of agreement between the observed and estimated values of a variable.

The integral square error is another statistical measure which describes the agreement between the time distribution of the observed and the estimated values of a variable. The smaller the value of the Integral Square Error is, the better is the agreement between the observed and the estimated values of a variable.

$$ISE = \frac{\left[\sum_{i=1}^{N} (O_i - C_i)^2\right]^{1/2}}{\sum_{i=1}^{N} O_i} \times 100$$

To compare the relative performance of each model, the ratings which have been assigned based on the values of these statistical measures are presented in Table 3.

A P P E N D I X I V

Comparative Analysis of Routing Models

APPENDIX IV

COMPARATIVE ANALYSIS OF ROUTING MODELS

With the advent of high speed computers and recent developments in numerical techniques, improved methods using more sophisticated hydraulic equations have been proposed for application to sewer flows. The two basic equations representing the gradually varied free-surface unsteady flow are the momentum equation

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + g \frac{\partial h}{\partial x} - g(S_o - S_f) = \frac{V.q}{A}$$

and the corresponding equation of continuity

$$\frac{\partial h}{\partial t} + \frac{A}{B} \frac{\partial V}{\partial x} + v \frac{\partial h}{\partial x} = \frac{q}{B}$$

Where

V = velocity

t = time

x = longitudinal coordinate along channel bottom direction.

g = gravitational acceleration

h = depth of flow

S_o = channel bottom slope

S_f = friction slope

q = the distributed lateral inflow (or outflow) as discharge per unit length of the conduit

A = area of channel cross section

B = water surface width

These are quasi-linear hyperbolic first-order partial differential equations derived from the well known St. Venant equation by adding the right hand side to each, which represents the effect of the lateral flow. These two equations can be solved numerically by using the method of characteristics. Although this method gives the most accurate of all practical methods of flood routing in channels and conduits, it requires a considerable amount

of computation and difficulties are also encountered in defining the boundary conditions. Hence, various simplifications have been proposed to give simple approximate solutions.

One approach which is known as hydrology routing is to use only the continuity equation. The hydrologic routing techniques, including the various coefficient routing methods such as the Muskingum technique and the reservoir routing technique, are used in some sewer flow models. Another approach is to solve the continuity equation together with various simplifications of the momentum equation. Among these are the kinematic wave approximation and the diffusion wave approximation. All the aforementioned flow routing techniques are applicable to a single sewer. When these techniques are applied to a network, the sewers or channels are simply treated individually in sequence with the flow cascading downstream from one channel to another. However, in a sewer network, considerations must be given to such mutual dynamic effects as backwater and energy losses among sewers and junctions.

Some of the available sewer flow models mentioned in this report are briefly analyzed in this appendix. The comparison is based on the following criteria:

(a) the routing technique;

(b) the degree of approximation used;

(c) the level of consideration of the backwater and energy losses in the junctions.

(i) <u>The SWMM of EPA</u>

The Transport Model of the EPA uses a simpler form of the St. Venant equations. The continuity equation is solved with the quasi-steady dynamic-wave approximation which is the momentum equation with the rate of change velocity term neglected. While an implicit scheme is used to solve the continuity equation, an explicit scheme has been adopted for the momentum equation. With the latter acting as auxilliary equation, a Newton-Raphson technique is used to solve the non-linear continuity equation. The friction slope is evaluated from Manning's formula, and the lateral flow is not simulated.

Like the MWIS model, no consideration is given to the backwater effects on the junction storage. However, lift stations, flow dividers

and reservoir routing techniques are modelled.

The model has been compared with the method of characteristics for a single line, but there is no indication about its accuracy when the backwater effect has a considerable role on the flow characteristics of a big system.

(ii) RRL Model

The RRL Model uses a hydrology routing technique in the transport sub-model, in which only the continuity equation is considered. The equation used is as follows:

$$\frac{\Delta t}{2}(I_i + I_2) = \frac{\Delta t}{2}(O_i + O_2) + S_2 - S_i$$

where I is the inflow, O is the outflow, S is the storage in the sewer and the subscripts indicate the volumes at the beginning and end of the time increment. A storage flow relationship has to be derived either by analyzing recorded hydrographs or by considering assumptions if measurements are not available. Backwater and junction losses are not considered.

The RRL transport sub-model underestimates the peak flow in a single pipe if it is compared to the method of characteristics solution at the St. Venant equations and because of its simplified approach, is valid only for small areas with short sewer reaches.

(iii) UCUR Model

The UCUR Model uses a simple time offset routing technique in all conduits and channels. The travel time is estimated by considering the average of the velocities for the flow depths in each time step, weighted for the volume of flow in that time step. An alternative procedure is to compute the velocity corresponding to the centroidal discharge of the hydrograph. Backwater and junction losses are not considered.

This method always overestimates the peak flow and eliminates the damping effect of system storage. Because of this, the method is not valid for large areas or long sewer reaches.

(iv) Improved SWMM developed by Water Resources Engineers, Inc., Systems (WRE)

The motion and continuity equations are used as follows:

$$\frac{\partial Q}{\partial t} = (-g\ A\ S_f) + (2V \cdot \frac{\partial A}{\partial t}) + (V^2 \cdot \frac{\partial A}{\partial x}) - (gA \cdot \frac{\partial h}{\partial t})$$

and

$$\frac{\partial h}{\partial t} = \frac{\Sigma\, Qt}{A_{st}}$$

Where $\Sigma\, Q_t$ = total inflow and outflow to the junctions at time t.

A_{st} = water surface area in the junction at time t.

The motion equation is applied to each link and the continuity equation to each node. Since an explicit approach by the use of the modified Euler method is used, there is a restriction on the size of the routing time step. If the time step is chosen higher than a certain limit, the solution becomes unstable.

Although this limit has been determined by Courant as

$$\Delta t \leq \frac{L}{\sqrt{gh}}$$

where Δt = time step

L = conduit length

it was found that a more appropriate stability relationship is

$$\Delta t \leq \frac{C^1\, As\, h_{max.}}{\Sigma\, Q}$$

Where C^1 is a constant determined experimentally as 0.1.

Although the model considers the entrance and exit losses in the conduits, it does not take backwater effects into account. It handles flow control devices such as weirs, pumps and tide gates. Surcharge conditions are solved by applying the first order correction based on the Hardy Cross method, which yields:

$$\left(\frac{\partial h}{\partial t}\right)_t = K\, K \frac{\Sigma\, Q_t}{g(\Delta t)^2 \Sigma \frac{A}{L}}$$

where K is a constant introducing some under-relaxation in the system, a value of 0.25 is used in the model.

A is the cross sectional area of the lines, and Σ is the summation of all lines entering node.

(V) Hydrograph Volume Method by Dorsch (HVM)

The St. Venant momentum and continuity equations in the form

shown in page 1 of this appendix are applied in each sewer reach.

The backwater effects are considered, which means that the sewer system is simulated as an interdependent network and the effect of every network element on the remaining elements is taken into account. The manholes, simulated as node points, are handled analogously by means of energy and continuity equations. Using an implicit scheme, the numerical solution of the entire equation system is carried out using an iterative technique. The model simulates lateral flow, but it does not consider manhole storage. It handles retention basins by applying a reservoir routing technique.

While the implicit solution scheme is more accurate and efficient than the explicit one, the method of characteristics gives the most accurate results. There is no indication about the difference in accuracy between the three methods, however, the computing time should be considered if any comparison is done.

The model was applied many times for practical systems and good results have been experienced upon verification.

(vi) Illinois Storm Sewer System Simulation Model (ISS Model)

The St. Venant equations for continuity and momentum are used. The lateral flow term is not included and it is assumed that inflow of storm water into the sewer system occurs only at discrete model points. This is of course justified for larger areas and equivalent watersheds. The equations are solved numerically by an explicit first order characteristic scheme. Backwater effects are considered and the reservoir type junction is modelled where manhole storage is taken into account. Energy and continuity equations are formed for each manhole and the entire equation system is solved simultaneously at each time step. The model handles circular sewers only and it assumes that there are no more than three sewers joined together at a manhole.

The model was tested for a typical case and the results look very promising. A user's manual is available and the authors are presently expanding the model to handle surcharge conditions.

(vii) Massachusetts Institute of Technology Model (MIT)

The model used the kinematic wave equations which are the continuity and an approximate form of the momentum equations which are:-

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q$$

$$Q = \alpha A^{m}$$

Where α and m are coefficients estimated from the Manning formula.

(Viii) High speed model developed by the Colorado State University (MWIS)

The Muskingum technique is used combined with an approximate form of the St. Venant momentum equation (inertia terms are neglected). With the routing coefficients changing each time step, some restrictions on the size of the routing time interval have to be employed in order to assure mass conservation.

Another restriction applies to the slope of the pipe which has to be significantly greater than zero, otherwise Muskingum's is not applicable.

The model does not take into account either backwater effects or junction storage. Junctions are considered as point nodes, and the hydrograph is routed through each single sewer, regardless of the effect of the downstream parts of the system.

Conclusion

The following Table I summarizes the comparison between the previously described routing models. With the fast progressing development in numerical techniques and high speed computers, there is no limit for sophistication. However, the effect of the degree of sophistication for simulating the flows, on the design and analysis of systems is still questionable.

For future analysis of the models, the HVM model can be considered as the most complex, when compared with the SWMM,the WRE and the ISS models. The last three models will be available through the University of Florida. The comparison should be based mainly on accuracy and computing time. It should be noted that neither conduit shapes nor the structures considered in each model should have a considerable weight in the comparison. Simple modifications can be added to include conduit shapes or structure types missing from any of the models.

TABLE I

Model	Routing Technique	Integration Scheme	Does the model include?		
			Backwater effects	Surcharge	Manhole storage
MWIS	Muskingum combined with the momentum equation with the inertia terms excluded		No	No	No
SWMM of EPA	Quasi-steady dynamic-wave approximation of the St. Venant equation.	Implicit for continuity explicit for momentum Newton Raphson	No	No	No
WRE	$\frac{\partial Q}{\partial t} = -gAS_f + 2V \frac{\partial A}{\partial t} - gA\frac{\partial h}{\partial x}$ $\frac{\partial h}{\partial t} = \frac{\Sigma Qt}{A_{st}}$	Explicit modified Euler method	No	Yes	Yes
HVM	St. Venant equation with lateral flow	Implicit scheme	Yes	Yes	No
ISS	St. Venant equations without lateral flow	Explicit first order characteristics scheme	Yes	No	Yes
RRL	Hydrologic reservoir routing technique		No	No	No
UCUR	Time offset technique		No	No	No

TABLE I (continued)

Design of sewers	Conduit shapes handled	Structures considered	Availability	Remarks
It has an optimization technique	Circular and Trapezoidal	None	Nonproprietory	
No	Circular, Semi-elliptical, egg shaped	Pumps, flow dividers, Internal storage units (weirs & orifices)	Proprietory	
No	Rectangular, circular horseshoe, basket-handle, eggshape, trapezoidal.	Weirs, pumps, tide gates, orifices	Proprietory	Studied presently by the University of Florida.
No	Any shape, by inputting the area-depth relationship	Retention rectangular basins, weirs	Proprietory	
Yes	Circular	None	Nonproprietory	Studies are carried on to add a surcharge subroutine
No	Any shape,by inputting the area-depth relationship	None	Nonproprietory	
No	Circular and rectangular	None	Nonproprietory	